'..... reverting again to the quotation from Matthew: "The Sage built his house and the winds and tides could not touch it because it was built on rock" (Book 6 - Verse 25), I was able to add: "But the Sage can build his house too, on any ground if he calls in the services of an injector"'.

Dr Claude Caron (1982)

THE GROUTING OF LOW PERMEABILITY SOILS IN DAM FOUNDATIONS

by David M Brett B.E., M.I.E. (Aust)

A thesis submitted in fulfilment of the requirements of the degree of Master of Engineering, at the University of Tasmania, 1985.

This Thesis contains no material which has been accepted for the award of a degree or diploma in any University, and to the best of my knowledge and belief, contains no copy or paraphrase of material previously published or written by another person, except where due reference is made in the text.

D Brett

....

David M Brett

30/1/86

Date

Installation of tubes a manchette for foundation grouting at the Refinery Catchment Lake Dam, Worsley Alumina Refinery, 1981.

Preface

In 1979 the author was appointed by his employers, Consulting Engineers, Gutteridge Haskins and Davey Pty Ltd as project engineer for the site investigation phase of a feasibility study into the management of fresh water resources and contaminated water at the proposed Worsley Alumina Refinery in the Darling Range of Western Australia.

He subsequently became project manager for the design phase of the $40 million "Water Management System" for which Gutteridge Haskins and Davey were responsible.

His total involvement with the project was completed when he was seconded to the Project Managers Raymond Engineers, Australia, Pty Ltd as an on-site technical consultant.

The Worsley "Water Management System" comprised three 'large dams', as defined by the International Committee on Large Dams (ICOLD) over twenty kilometres of diversion channels, various hydraulic structures, lined water storage basins, tailings disposal areas and various seepage collection systems.

A particular feature of the project was a multi-backup environmental protection system to control water contaminated by caustic soda and other chemicals used on the site.

The first level of protection involved the construction of a storage reservoir, called the Refinery Catchment Lake, to the highest practical degree of water tightness.

Part of the design for this reservoir comprised the incorporation of a grout curtain beneath the earth fill embankment in the thick weathered in-situ laterite soils.

Field testing confirmed that only low viscosity chemical grout could effectively reduce the already low permeability of the foundation,

Chemical grouting of soils is a relatively recent engineering development, and whilst a certain amount of experience has been gained, particularly over the past twenty five years, the procedures to be used are far from being precisely defined.

Practical field testing at the Refinery Catchment Lake Dam Site indicated potential problems with some grouting procedures used at other sites in the past. These procedures related principally to the grout pressures and injection volumes used with evidence that excessive hydraulic fracturing of the low permeability soil was counter productive to the overall aim of economically reducing permeability.

The thesis describes the design of the Refinery Catchment Lake Dam, reviews the history and theory of chemical grouting and discusses some relevant case histories. It then describes the development of a practical technique used to inject cement/bentonite grout and a phenoplast grout, Geoseal MQ4, into the dam foundations to achieve the design requirements.

The thesis was prepared following post-construction research by the author to deepen his knowledge on the general subject of chemical grouting.

The thesis was written in the belief that the work carried out at the Refinery Catchment Lake Dam was unique in terms of the attempted reduction of already low permeability. It is believed that in documenting a major field operation the thesis makes a positive contribution to the increase in knowledge of low permeability soil grouting techniques and points to areas where additional academic research is required.

ACKNOWLEDGMENTS:

Many people have helped and influenced my work towards this thesis, firstly in association with the original project work and secondly in the preparation of the text presented here.

Special acknowledgment is due to: my employers Gutteridge Haskins and Davey Pty Ltd, particularly Mr John Phillips, Director, for this faith in my ability, often far beyond my own; my associates, Dr Glen Truscott for his calm logic and, David Coheny for his invaluable preliminary research; Project Managers, Raymond Engineers, particularly Kas. Sobejco whose support of technical requirements over contractural expediency was probably the major influence on the technical success of the project; the Contractor, GFWA Pty Ltd, particularly Trevor Osbourne, for his practical advice; Worsley Alumina Pty Ltd, particularly the project liaison officer, Jim Irish for his interest in the project and his personal encouragement towards the preparation of this thesis, Staff of the University of Tasmania, particularly Dr Greg Walker for his early advice; my supervisors Brian Cousins, Bruce Cole and Mike Fitzpatrick for their constructive criticism and guidance; Lynne and Shelley for their translation of my manuscript and their wonderful typing, and lastly my wife, Margaret and my family who have been patiently supportive over the past two years.

NOTATION

A	=	area (m^2)
A_f	=	Skemptons pore pressure, parameter
a_1	=	effective spherical radius of a grout source (m)
a_2	=	distance of grout front from source
B	=	width of dam base
C	=	a shape constant
D_1	=	depth of overburden (m)
D	=	depth of grout curtain (m)
d_s	=	effective diameter of a soil particle (m)
E	=	Youngs modulus of elasticity
E_c	=	efficiency of a grout curtain
e	=	void ratio of soil
h	=	applied hydraulic head (m of water gauge)
h_i	=	grout injection head (m of water gauge)
i	=	hydraulic gradient
K_o	=	co-efficient of horizontal earth pressure
k	=	Darcy's co-efficient of permeability (m/sec)
k_o	=	Cozeny-Carman factor
k_g	=	apparent permeability of soil to grout (m/sec)
k_h	=	horizontal permeability (m/sec)
k_v	=	vertical permeability (m/sec)
L	=	length (m)
L_i	=	length of injection hole (m)
N	=	borehole expansion pressure ratio
n	=	porosity of soil
P_o'	=	effective overburden pressure (Pa)
P	=	grout pressure (Pa)
p_o	=	internal pressure in grout hole (Pa)
Q	=	flow rate (m^3/sec)
q	=	grout flow rate (m^3/sec)
R	=	radius of grout front at time t. (m)
R_p	=	effective soil pore radius (m)
r	=	radial distance from grout hole centre (m)
r_o	=	radius of grout hole (m)
r_1	=	radius of plastic zone (m)
S	=	specific particle surface area (m^2)

St = soil tensile strength (Pa)
t = time (secs)
v = velocity of flow (m/sec)
W = width of a flaw in a grout curtain
Z = number of flaws in a grout curtain
γ = bulk density (N/m^3)
γ_g = density of grout (N/m^3)
γ_w = density of water (N/m^3)
μ = dynamic viscosity (Pa.s = 10^3cP) *
μ_g = viscosity of grout (Pa.s)
μ_{pl} = plastic viscosity (Pa.s)
μ_w = Viscosity of water (Pa.s)
ε_s = shear strain (m)
ν = Poissons ratio
σ_r = radial stress from soil forces (Pa)
σ_t = tangential stress from soil forces (Pa)
σ_z = vertical stress from soil forces (Pa)
σ_R = radial stress from seepage forces (Pa)
σ_T = tangential stress from seepage forces (Pa)
σ_Z = vertical stress from seepage forces (Pa)
τ = shear strength (Pa)
τ_f = undrained shear strength (Pa)

* Note:

The correct unit for viscosity in the SI system is Pa.s. The unit centipoise (cP) equal to 10^{-3}Pa.s is used in the text due to its common use in the grouting field.

THE GROUTING OF LOW PERMEABILITY SOILS IN DAM FOUNDATIONS

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CHAPTER 1

INTRODUCTION

This thesis deals with the engineering practice of grouting, defined by Bowen (1981) as "... the injection of appropriate materials under pressure into certain parts of the earths crust through specially constructed holes, in order to fill, and therefore seal, voids, cracks, seams, fissures, or other cavities in soils and rock strata".

The grouting of rock strata using cement slurry is a well known technology with a documented history of almost two centuries.

In his keynote address to the Conference on Grouting in Geotechnical Engineering, held in New Orleans, February 1982, Adam Clive Houlsby went so far as to suggest that experienced personnel "can nowadays carry out cement grouting as an engineered process in order to achieve specified standards of workmanship".

Unfortunately the same cannot yet be said of the chemical grouting of soil foundations.

The first documented case of successful grouting of soils by chemicals was by Joosten in 1925. Since then, particularly in the past twenty five years, the technology of chemical grouting has made dramatic advances. The range of chemicals available and the infinite variability of natural soils, however, makes the subject of soil grouting extremely complex.

The documentation of completed projects is not extensive and, particularly in Australia, the level of experience of individuals or organisations is not great.

This thesis reviews the history, theory and practice of chemical grouting of soils, particularly as related to the construction of grout curtains in dam foundations.

A significant portion of the thesis describes the design of a grout curtain for a major dam at the Worsley Alumina Refinery in Western Australia and the subsequent development of practical grouting techniques used on the project.

CHAPTER 2

THE WORSLEY "WATER MANAGEMENT SYSTEM"

2.1 INTRODUCTION

The Worsley Alumina Refinery is located on the headwaters of the Augustus River, about 200km south of Perth in the Darling Range of Western Australia. The area is indicated in Figure 2.1.

Detailed engineering investigations and design for the work commenced in 1979 with construction being completed over the period 1981 to 1984.

The refinery processes bauxite by the Bayer process. This process involves the extraction of alumina from the ore using a hot, concentrated caustic soda solution. Approximately three tonnes of ore are processed to produce one tonne of alumina. The remaining 'bauxite residue' is filtered, to remove the majority of the caustic liquid, and is disposed of on site by a dry stacking technique.

Stormwater run off from the site and leachate from the bauxite residue stacks is expected to contain a proportion of residual caustic soda and other chemicals which would lead to an increase in pH and salinity in the receiving waters if not controlled. As the Augustus River is claimed to be a particularly low salinity water source and one of the few remaining developable water resources in the south west of Western Australia, the environmental protection of the river was a major factor in the State Government's approval of the scheme. The developers were legally committed to fully utilise the extent of modern engineering technology to achieve complete control. The resulting "Water Management System" comprised a system of dams, channels and groundwater bores designed to collect and contain any contaminated water whilst bypassing the maximum volume of uncontaminated water.

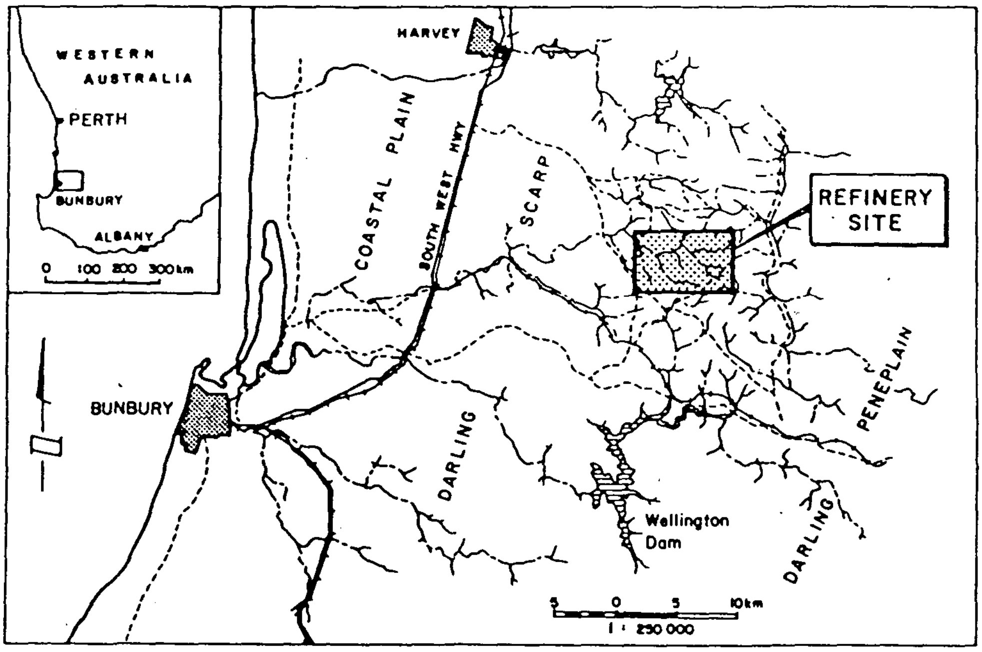


Figure 2.1. Worsley Alumina Refinery - Location

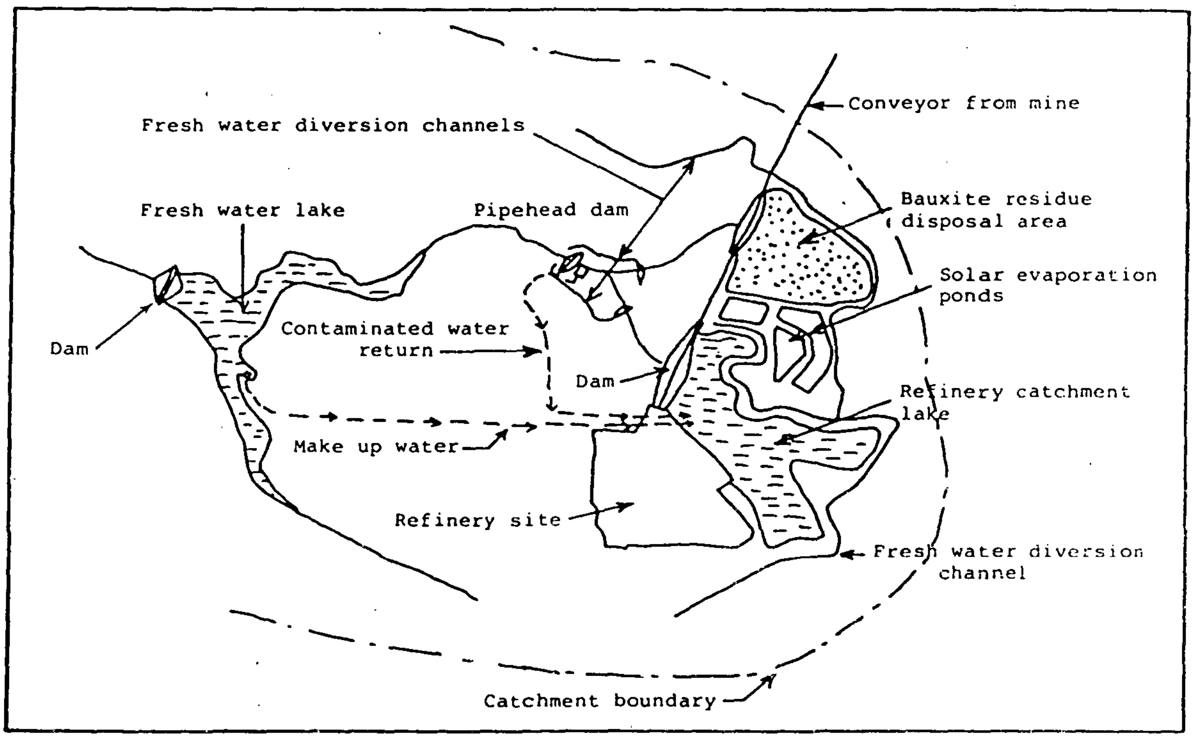


Figure 2.2. Worsley Water Management System

The author was associated with the investigations, design and construction phases of the water management system through his employment with the consulting engineers, GHD Dwyer Pty Ltd, a subsidiary of Gutteridge Haskins and Davey Pty Ltd.

This company was responsible initially for feasibility studies for the major shareholders in the venture, Reynolds Australia, Alumina Pty Ltd and later for design, as consultants to the Project Construction Managers, Raymond Engineers, Australia Pty Ltd.

2.2 THE WATER MANAGEMENT SYSTEM

A plan of the refinery site indicating the essential features of the water management system is shown in Figure 2.2

The site is located in one of the highest rainfall areas in Western Australia, with the average annual rainfall of 1350mm being only slightly lower than the annual evaporation of 1490mm (Gutteridge Haskins and Davey, 1980).

TABLE 2.1 shows the monthly rainfall and evaporation figures.

Table 2.1
Monthly rainfall/evaporation - Worsley Refinery

Month	Average Rainfall (mm)	Average Evaporation (mm)
January	14	220
February	16	200
March	30	160
April	58	85
May	187	60
June	278	55
July	268	65
August	205	70
September	139	75
October	98	110
November	40	160
December	19	230

The separation and disposal of uncontaminated surface run off water is critical to the success of any control system for contaminated water.

The major sources of contamination are run off from the refinery site itself and run off and seepage from the bauxite residue disposal stacks. Contaminated water is collected and discharged directly by gravity or indirectly by pumping, via the Pipehead Dam, to the Refinery Catchment Lake.

This lake, which is situated beside the refinery process area serves as both a storage for contaminated water and as a cooling pond for the refinery's coal burning power station. The use as a cooling pond results in a significant increase in evaporation, enabling, with maximum diversion of fresh water, operation of the lake at a stable water level. The lake has been sized with sufficient freeboard such that even the most severe conceivable combination of rainfall/evaporation events would not lead to any spillage.

During drought periods top up of the Refinery Catchment Lake can be effected by pumping from the Fresh Water Lake, a 6000 Ml storage located downstream.

Prevention of contamination of surface water or groundwater is ensured in the system design by:

a) Maintenance of pre-construction groundwater flow lines towards to Refinery Catchment Lake.

b) Minimisation of seepage from the Refinery Catchment Lake.

c) Collection of seepage from the Bauxite Residue Disposal Areas by a patented drainage system which separates and controls groundwater and leachates.

d) Maintenance of a water table "sump" at the Pipehead Dam as a 'secondary' collection system for any seepage bypassing primary controls.

e) Location of a fresh water supply downstream of critical areas which indirectly ensures the refinery's owners interest in proper management of the system.

f) Regular monitoring of surface water and groundwater status at dozens of measurement points and boreholes around the site.

The system is described fully in GHD Dwyer's successful submission to the Institution of Engineers, Australia, WA Division for their Engineering Excellence Award of 1983, attached as Appendix A.

The project was also successful in its submission to the WA Chapter of the Association of Consulting Engineers, Australia, for their 1984 Excellence Award.

The dams and structures have been described by Truscott and Brett (1983).

This thesis deals in detail with Item (b), the minimising of seepage from the Refinery Catchment Lake Dam.

2.3 SITE GEOLOGY AND SOIL CHARACTERISTICS

The site geology and soil conditions have been described by Gutteridge Haskins and Davey (1980), Gordon (1984) and Gordon and Smith (1984).

The rock types found in the Darling Range are granites, quartzites and gneisses, often with instrusions of dolerite dykes and quartz seams. Much of the rock has been subject to intense shearing and is deeply weathered due to the combined effects of high rainfall, high evaporation, high water tables and the extent of vegetation. Depths to bedrock in the Worsley area have been recorded at over forty metres.

The soil profile conforms generally to the classical laterite model with ferruginous, mottled, pallid and zersatz ('rotten rock'), zones described by Gordon.

Two distinct sub groups of weathering profile are reported by Gordon, being the 'plateau' profile (developed with relatively low water table) and the 'valley' profile (developed with relatively high water table). Figure 2.3 shows the various profiles derived by Gordon with notes on the characteristic properties of the different soils.

Gordon and Smith (1984) summarised the various engineering properties of the most significant soil type, the pallid zone.

The results were obtained from numerous laboratory and insitu tests related to dam site investigations and the refinery process plant site investigations.

Atterberg limits indicated predominantly ML and MH materials with generally higher plasticity including some CH soils from weathered dolerite. Plasticity charts are presented in Figure 2.4.

GENERAL CLASSIFICATION	GEOLOGICAL MATERIALS	LOG	ENGINEERING PROPERTIES
Soil	Soil—Sand		Poor borrow
FERRUGINOUS ZONE Laterite (totally weathered granite)	Pisolitic Laterite		High permeability, excellent borrow material
	Massive Laterite		Unsuitable for borrow - strongly cemented or bouldery
MOTTLED ZONE	Gibbsite Laterite		Excellent borrow material (permeable)
PALLID ZONE	Kaolin - Gibsite Laterite		Good borrow-impermeable
Highly Weathered Granite	Kaolin Phase sequi-oxide accummulation (may contain red ferruginous nodules)		Fair borrow material (impermeable) Good foundation material Low Permeability
ZERTSATZ ZONE	GWL-Ferruginated Zone		Permeable - recemented
	Quartz Residual		Permeable - poor borrow material Little cohesion
	Mica Residual		Found above more micaceous rocks - deleterious borrow material
Moderately Weathered Granite	Moderately weathered		Moderate to strong foundation rocks
Slightly Weathered Granite	Slightly weathered		Moderately strong to very strong foundation rock
Fresh granite stained joints	Fresh rock with limonite-stained joints		Strong to extremely strong foundation rock
Fresh Granite	Fresh rock		

PLATEAU LATERITIC WEATHERING PROFILE OF NORMAL GRANITIC ROCK

GENERAL CLASSIFICATION	GEOLOGICAL MATERIALS	LOG	ENGINEERING PROPERTIES
Black soil, red laterite ironstone gravel	A. Horizon soil & pisolitic laterite		Stony, soft, sensitive to moisture, high permeability
Red gravelly clay angular rubble	B. Montmorillonite to Kaolin Zone		Cracking clays, reworked strength poor, thin zone, Swells when wet
Khaki brown & red mottled clay, red blocks set in khaki clay becoming more friable & sandy with depth	C. Highly weathered		Minor soil activity kaolin replaces montmorillonite, a few residual rock cores, angular blocky pieces of sandy clay or clayey sand. Relict rock joints present Low permeability
GWL Fluctuation	Ferruginated Zone		Cemented - cherty ferruginous blocks
Red & brown clayey blocks, increasing content of dolerite cobbles	Soft rock highly to moderately to weathered		Jointed soft rock, weak to moderately weak some rock cores.
Dark green weathered-fresh dolerite boulders in clay	D. Moderately to slightly weathered		Firm to hard clay in joints, medium strong to strong rock joints filled with clay. Impermeable
Fresh dolerite	Jointed rock mass with sheet & cooling joints		Very strong to extremely strong brittle rock, jointed, highly permeable in joints

PLATEAU LATERITIC WEATHERING PROFILE OF DOLERITE ROCK

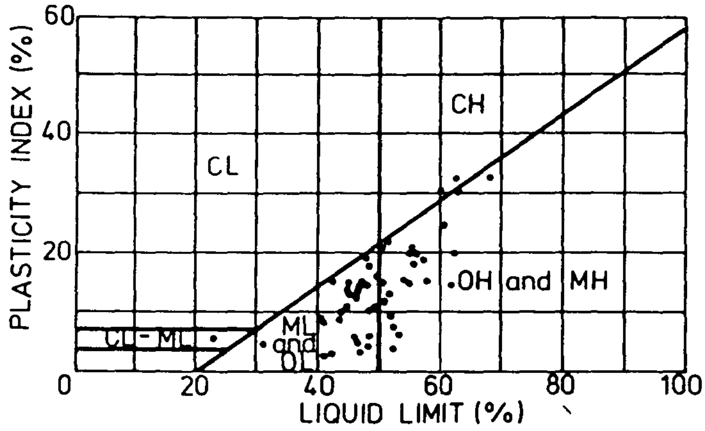
GENERAL CLASSIFICATION	GEOLOGICAL MATERIALS	LOG	ENGINEERING PROPERTIES
Gravelly clay, sandy gravel yellow brown	Pisolitic Laterite		Loose to soft, good borrow. High permeability
Red cemented sandy clay MOTTLED ZONE	Lateritic Clay		Firm, good borrow Low permeability
White, rusty stained in bands sandy clay - fissured	Modified kaolin zone with root fissures		Stand up time poor because of fissure joints, reasonable borrow material, permeable in fissures
White to banded stained sandy clay with boulders or core stones with spheroidal weathered PALLID ZONE	Kaolin Zone - core stones formed by sheet joints		Difficult foundations with isolated granite boulders spheroidally weathered sheet joints completely weathered
White to blue & gold sandy silt & micas with core stones ZERSATZ ZONE	Quartz - Mica, residual with core stones		Poor to deleterious foundation material. Stand up time 30 seconds when saturated
Granite rock mass	Banded porphyritic granite, few joints		Strong to very strong, few joints, some pegmatite & quartz veins

VALLEY LATERITIC WEATHERING PROFILE OF GRANITIC ROCKS

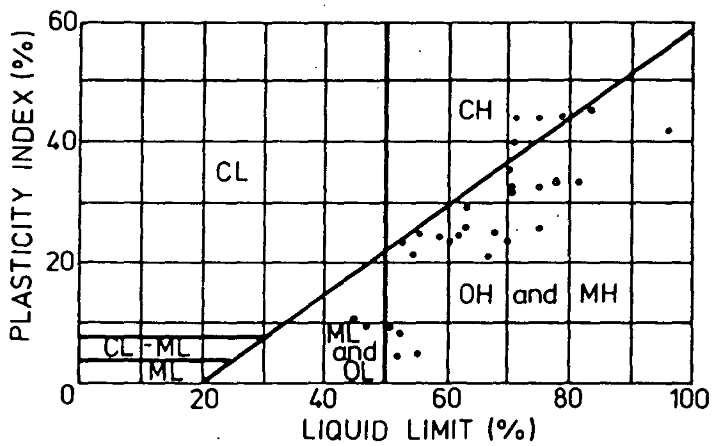
GENERAL CLASSIFICATION	GEOLICAL MATERIAL	LOG	ENGINEERING PROPERTIES
Black soil, rubble	A. Horizon		High permeability, soil properties
Red gravelly clay	B. Horizon Pisolitic clay & lateritic soil		Cracking active clays, more dense with depth, core stones, cobbles, high permeability
Red & brown gravelly sandy clay, friable with depth	Completely weathered dolerite C. Horizon		Bottom of zone of seasonal fluctuation of water table, soil becomes stable Low permeability
Rusty red & brown with yellow bands areas of cementation with black iron oxide	Dolerite completely to moderately weathered 'Rotted Rock' Core stones		Friable with little cohesion between minerals, mostly black iron oxide, weak rock substance. High permeability Spheroidal weathering
Black & brown patches with yellow bands, rock cores usual & increase in size & number with depth	Dolerite, Highly weathered		Friable to very stiff clayey sand, joints show & rock cores increase in size & freshness. Permeable
Dolerite rock mass	Slightly weathered jointed dolerite		Strong to very strong rock. Rock joints permeable.

VALLEY LATERITIC WEATHERING PROFILE OF DOLERITE ROCK

Figure 2.3. Worsley Area - Laterite Weathering Profiles (after Gordon)



PLASTICITY CHART - GRANITIC CLAYS & SILTS



PLASTICITY CHART - DOLERITIC CLAYS & SILTS

Figure 2.4. Worsley Area Soils
Plasticity Charts - Pallid Zone

The extreme variability of the laterite soil types is indicated by the variation of dry density between 1.05 and 1.76 tonnes per cubic metres, and moisture content, from 10% to 60%. A trend for decreasing dry density with depth was noted. These results are presented in Figure 2.5.

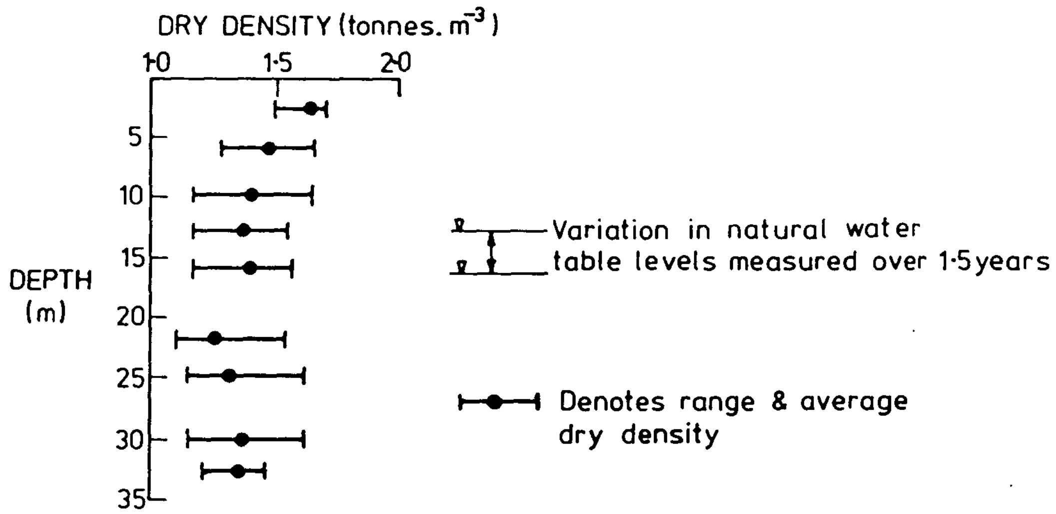
Peak shear stresses measured by Camkometer ranged between 214 and 806 kPa with an average of 429 kPa. The Camkometer, described by Wroth and Hughes (1973), is a highly sensitive instrument developed at Cambridge University for measuring lateral stresses, undrained stress-strain properties and drained peak shear stresses of clay. The instrument consists of a hollow cylinder which can drill itself into the ground with a minimum of disturbance. The unit used is operated by the University of Western Australia. A general trend of decreasing shear strength with depth was noted. This was explained by Gordon as related to the reduction in the proportion of cementitious sesquioxides which give laterite soils their characteristic quasi-overconsolidated parameters.

Lateral earth pressures measured with the Camkometer are plotted in Figure 2.6.

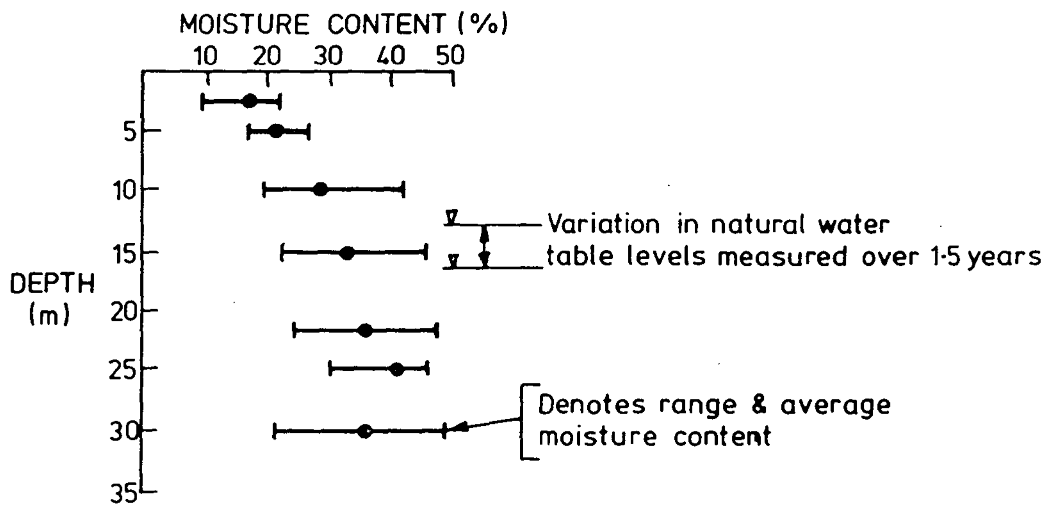
There is no apparent correlation between lateral earth pressures and overburden pressure with K_o varying from approximately 0.35 to 2.5.

Parameters measured in over 30 oedometer tests included calculation of overconsolidation ratios from 1.2 to 4.0 with an average of 2.7 and standard deviation of 0.6.

The modulus of elasticity (E) was calculated at from 7 to 42 MPa with an average of 26. Poisson's ratio (ν) was found from Camkometer testing to average 0.4.



DISTRIBUTION OF DRY DENSITIES WITH DEPTH



DISTRIBUTION OF MOISTURE CONTENTS WITH DEPTH

Figure 2.5. Worsley Soils
Density/Moisture Relationships

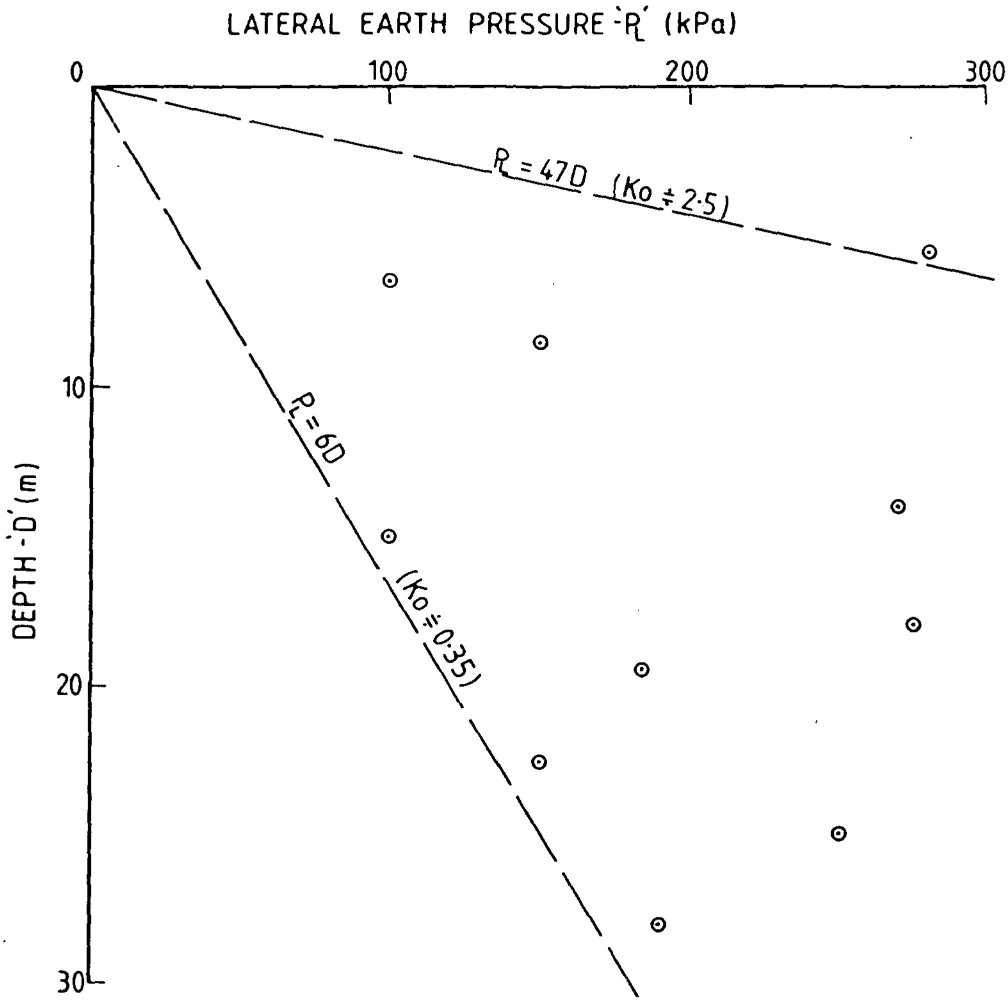


Figure 2.6. Lateral Earth Pressures vs Depth
(from camkometer testing)

2.4 THE REFINERY CATCHMENT LAKE DAM - DESCRIPTION AND GEOLOGY

The dam comprises an earthfill embankment approximately 600 metres long with a maximum crest level approximately 25 metres above natural creek level.

The embankment cross section as shown in Fig. 2.8 is characterised by an upstream inclined core. Outer earth shells both up and downstream are armoured by rockfill to prevent erosion.

A key trench under the core penetrates the surface soils to a depth of up to 4 metres.

Internal drainage is provided by an inclined chimney drain, a downstream sand/gravel blanket drain and relief wells.

As described earlier, the water stored in the lake will eventually become caustic and highly saline. The dam design includes no spillway or scour pipe since the evaporation, enhanced by the cooling pond effect, is sufficient to match inflow into the lake.

Site investigations included bulldozer and excavator trenching together with diamond drill holes, friction cone probe holes and geophysical investigations. A number of falling head permeability tests were carried out as were several lateral earth pressure tests with a borehole pressuremeter. A cross section across the valley on the dam axis indicating geological features and test results is shown in Figure 2.7.

The geology of the site was found to be typical of the refinery area as described previously. Bedrock was granite with dolerite intrusions, with the weathered rock/soil profile varying from 10 to 40 metres thick. Weathering conditions ranged from 'plateau' type on the abutments to 'valley' over the lower valley floor.

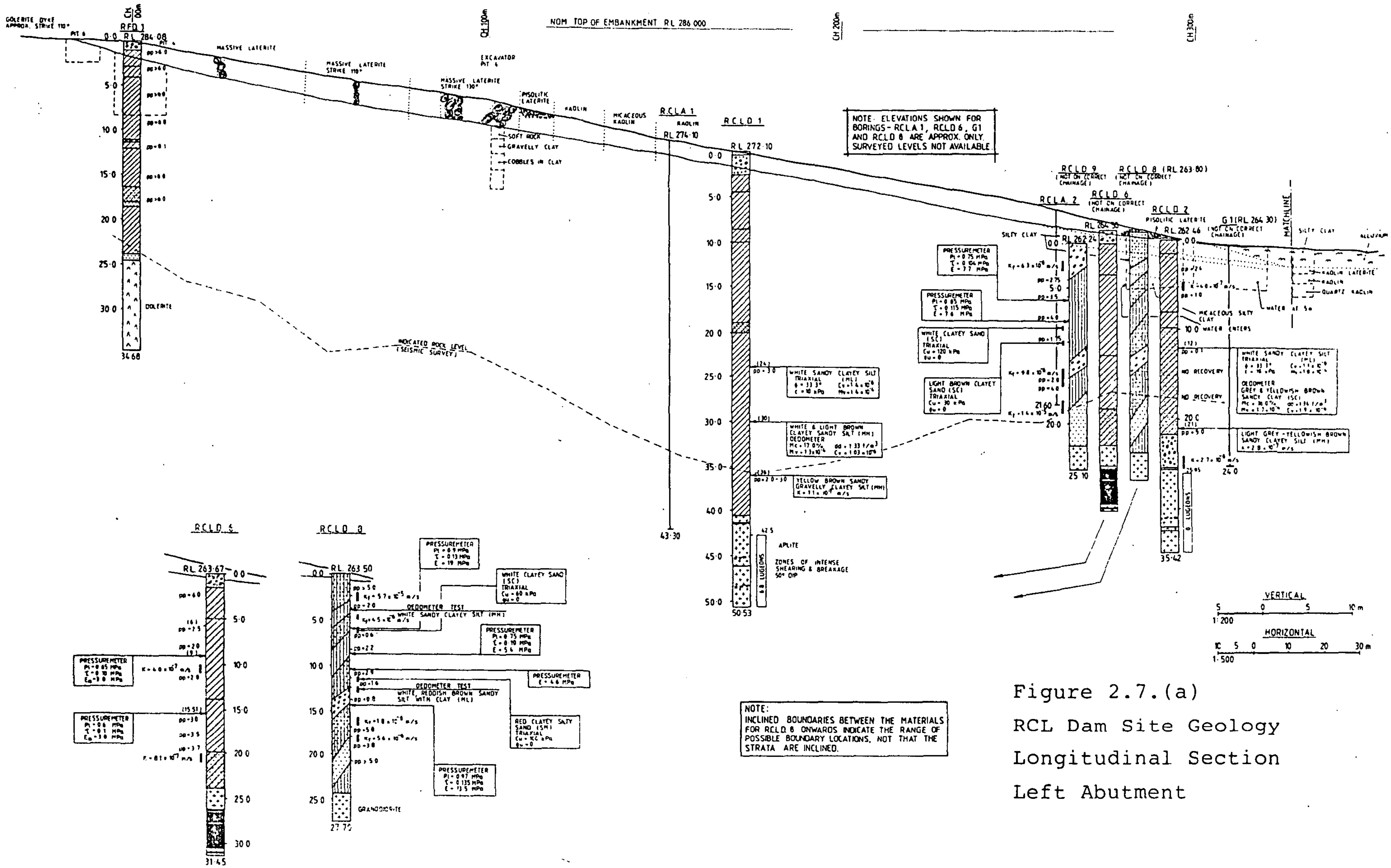


Figure 2.7.(a)

RCL Dam Site Geology

Longitudinal Section

Left Abutment

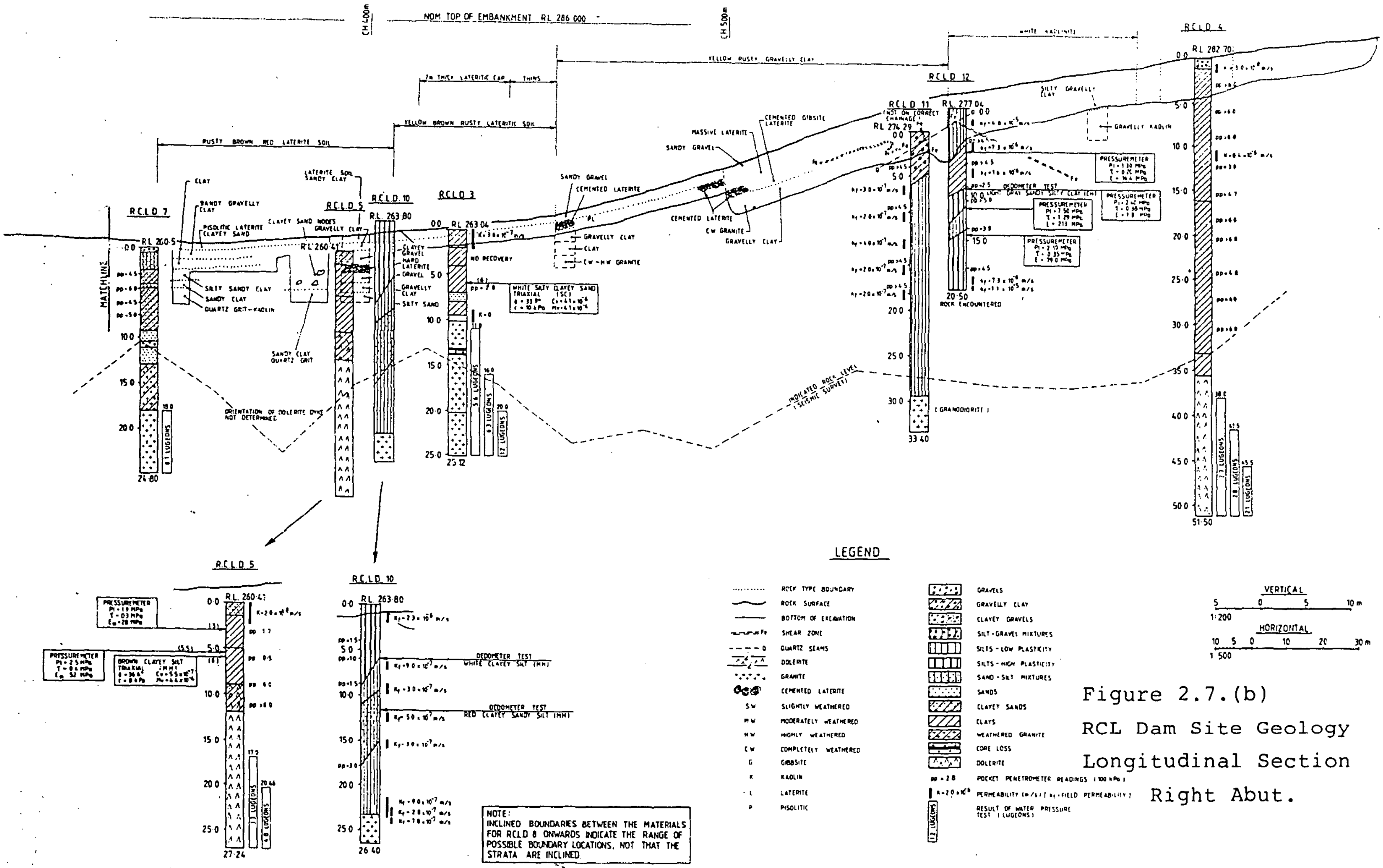


Figure 2.7.(b)

RCL Dam Site Geology

Longitudinal Section

Right Abut.

Permeability testing indicated that the foundation soils were reasonably tight with the majority of tests indicating permeabilities less than 10^{-6} metres/second. However, several tests indicated moderate permeability, around 10^{-4} metres/second, associated with sandier materials particularly around the bedrock/soil interface (zersatz or quartz/mica residual). This zone was exposed in various areas adjacent to the dam site where bedrock was close to the surface and was noted to be water leaking. The permeability relationships of the soil profile suggested by Gordon (1984) were not convincingly demonstrated; however, within the limitations of the investigations, there was evidence that more permeable bands could exist through the foundations.

2.5 THE REFINERY CATCHMENT LAKE DAM - SEEPAGE ANALYSIS

To assess the effects of seepage through and under the dam a detailed two dimensioned seepage analysis was carried out using a finite element computer programme, RESEP, developed at Sydney University. The programme was expanded to enable a mesh of 622 nodes. This was necessary to enable the study of combined seepage through both the foundation and the embankment.

For the purpose of seepage analysis, features of the foundation were modelled as:

1. Impervious rock at average 20 metres.

2. Cut-off trench to 3 metres intercepting surface permeable features.

3. Optional one metre thick sand layers at depths of 9 metres and at 19m (above rock level).

4. Optional single row (3m wide) or double row (5m wide) grout curtain.

These features are shown in Figure 2.8.

The cross section adopted for analysis corresponded to the cross section of the dam at maximum height and the inferred foundation conditions at the same location. These foundation conditions were considered to be the worst likely conditions that could be encountered along the entire dam length. For these reasons, it was necessary to model one critical section only.

As the hydraulic head would be lower for sections further up the abutments, seepage per unit wall length will decrease at these sections. It was conservatively estimated that total seepage quantities would be equivalent to 350 metres width of the adopted profile.

Features included in the mesh used were as shown in Figure 2.9.

1. Provision for two upstream zones (ie, core and upstream shell).

2. An inclined chimney drain of variable width downstream of the core. The chimney drain can be up to 3m and three elements wide.

3. No downstream shell. The high capacity of the inclined chimney drain obviates the need for the downstream shell to be modelled. As a result, many unnecessary nodes and elements are eliminated.

4. A horizontal blanket drain of up to 1.8m maximum thickness comprising five elements width which allow for a gravel drain of three elements width sandwiched between two sand filter layers.

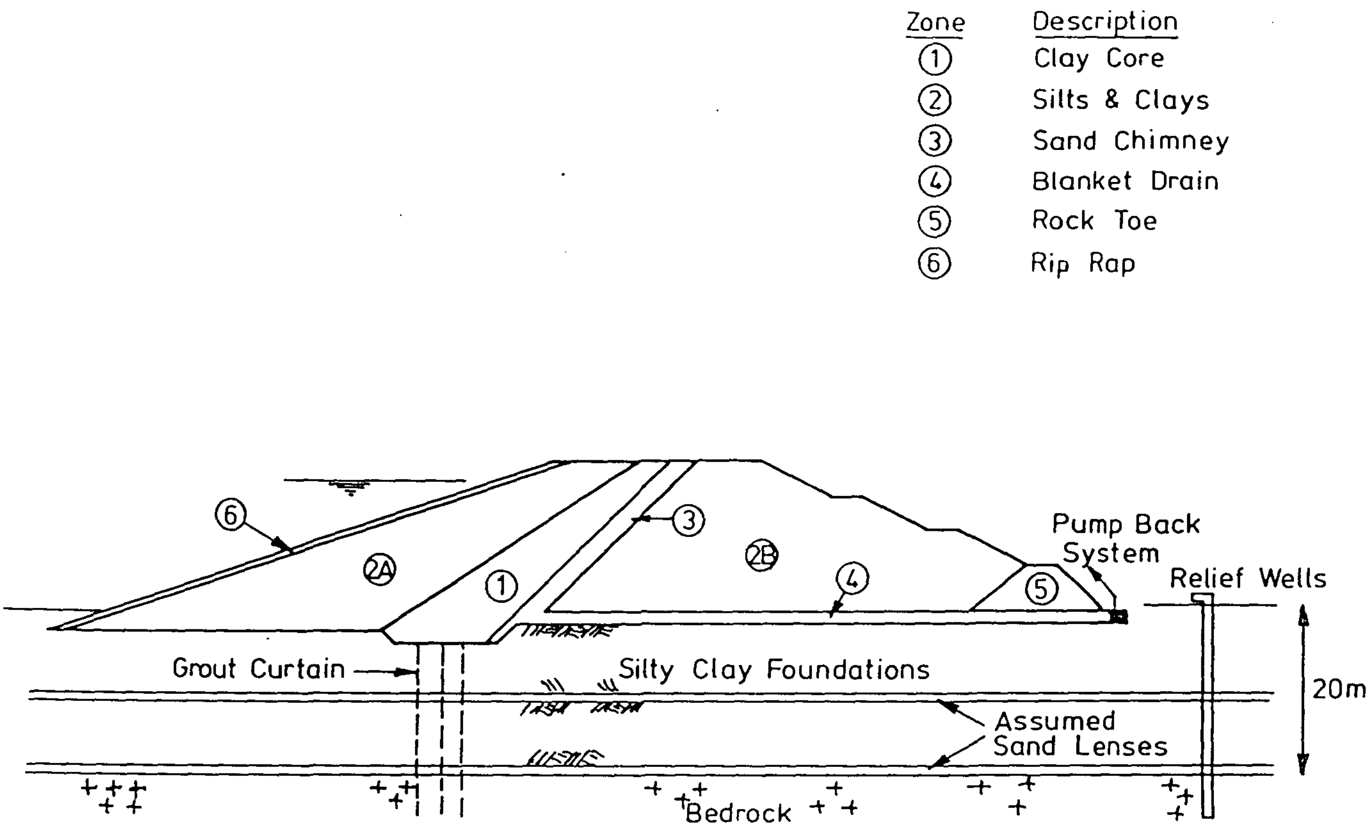


FIG. 2·8 R.C.L. DAM - TYPICAL X-SECTION
FOR SEEPAGE ANALYSIS

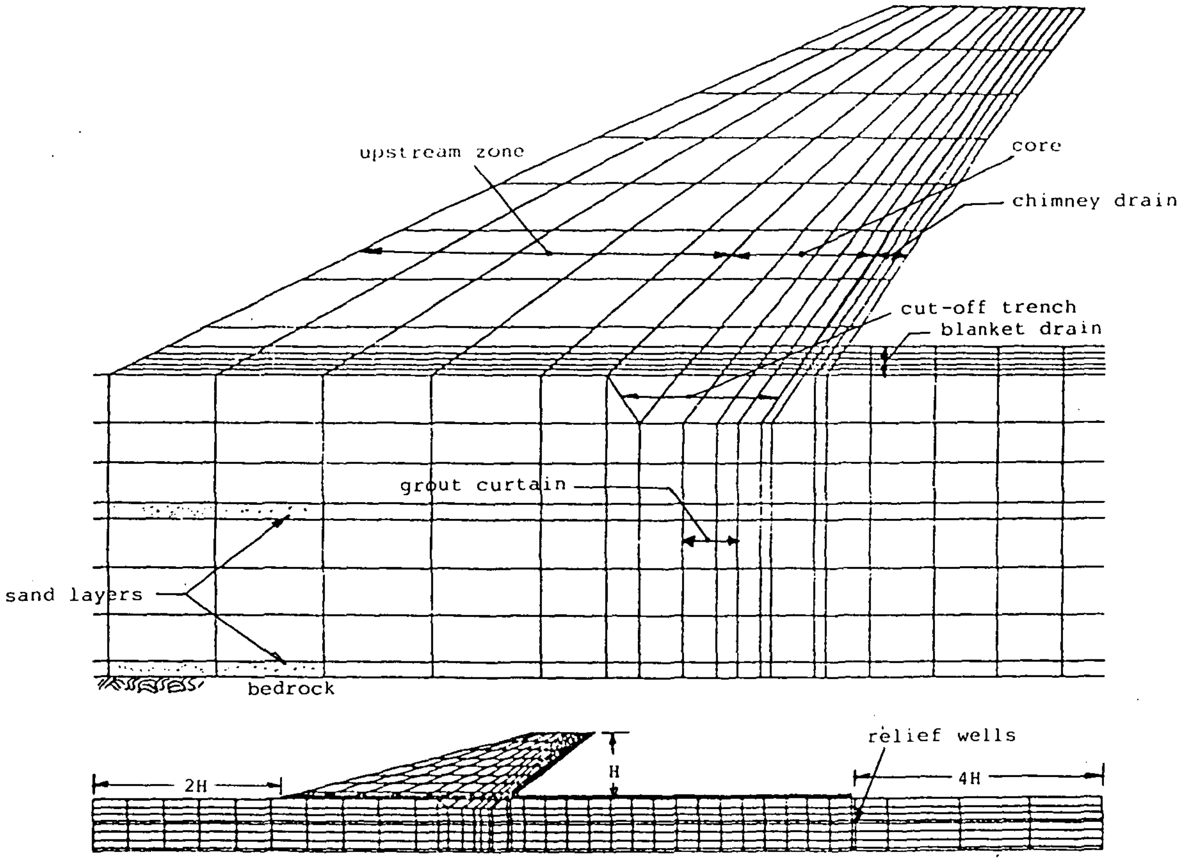


Figure 2.9. Finite Element Mesh

5. The foundation extends twice the hydraulic head upstream of the dam and four times the hydraulic head downstream of the dam. Runs were also performed with the foundation extending four times the hydraulic head upstream of the dam, but this had no effect on the solution for total flow.

6. Sand layers in the foundation at depths of 9 metres and 19 metres, and each 1 metre thick.

7. Impervious rock at 20 metres depth.

8. Cut-off trench to a depth of 3 metres below stripped level.

9. A grout curtain of 3 metres or 5 metres width extending from the cut-off to bedrock.

10. Relief wells downstream of the dam.

Any of these features could be introduced or deleted from the geometry as required, by adjusting the permeabilities of the relevant elements.

Permeabilities of each zone were based on the likely range of values and the most probable values, as determined by in-situ permeability testing on all foundation materials, and by laboratory falling head permeability tests on undisturbed embankment material. The permeability of embankment drains was inferred from the particle size distribution of the relevant material using the Hazen formula.

The adopted permeabilities were:

Embankment Core	$k = 1 \times 10^{-9}$ m/s
	$k = 1 \times 10^{-8}$ m/s
Embankment Upstream Zone	$k = 1 \times 10^{-8}$ m/s
	$k = 1 \times 10^{-7}$ m/s

Gravel in Blanket Drain $k = 1 \times 10^{-2}$ m/s

Sand in Blanket Drain and
Inclined Chimney Drain $k = 5 \times 10^{-4}$ m/s

Foundation Clayey Silts $k = 1 \times 10^{-6}$ m/s

Sand Layers in Foundation $k = 1 \times 10^{-4}$ m/s

The permeability of the foundation sand was higher than the average measured value to allow for variations in layer thickness and permeability, and to ensure conservative results.

To assess the effect of grouting of soils it was assumed that sands could be successfully penetrated by grout, resulting in very low permeabilities, whereas, the effect on the less pervious clayey silts would be to reduce their permeability by a much lower amount. Inspection of undisturbed samples of foundation materials, and, exposures in excavated cuttings suggested that the major cause of permeability in the clayey, silty soils would be fissures, root holes, quartz seams and other features. Thus it was predicted that grouting would have a significant effect even in these materials.

Permeabilities, used in the analysis, were:

Grouted Sands $k = 1 \times 10^{-8}$ m/s

Grouted Clayey Silts $k = 2 \times 10^{-7}$ m/s

The bedrock was assumed to be impermeable and located at an average of 20m depth. An examination of the flow nets indicated that any seepage at depth is relatively small, and even if the rock permeability was of the same order as the foundation soils, the results would not have been affected by more than 10%.

The foundation cases analysed are described below:

a) Foundation assumed homogenous and isotropic.

b) Foundation is assumed to have a one metre thick sand layer overlying the rock.

c) Foundation is assumed to have two sand layers - one metre thick overlying the rock, and one metre thick ten metres below the natural surface.

Analyses were performed for grout curtain widths of both 3 metres and 5 metres.

Results are summarised in Table 2.1.

Table 2.1
Results of Seepage Analysis

FOUNDATION CONDITIONS ASSUMED	CALCULATED SEEPAGE (Ml/year)					
	NO GROUTING		SINGLE ROW CURTAIN		MULTIPLE ROW CURTAIN	
	TOTAL	BYPASS	TOTAL	BYPASS	TOTAL	BYPASS
Homogeneous, Isotropic foundation $Kc = 10^{-6}$m/sec $Kgc = 10^{-7}$m/sec	63	1	45	1	38	1
Sand layer over rock $Ks = 10^{-4}$, $Kgs = 10^{-8}$m/sec $Kc = 10^{-6}$, $Kgc = 10^{-7}$m/sec	213	18	77	5	56	3
2 sand layers K as above	406	40	100	8	67	6

NOTE: Bypass means seepage flow escaping other collection systems at the downstream tow of the dam. Ks denotes permeability of sand, Kgs denotes permeability of grouted sand. Kc denotes permeability of silty clay, Kgc denotes permeability of grouted silty clays.

A homogeneous foundation allowed only a small amount of seepage under the core with negligible seepage remaining in the foundation beyond the downstream toe of the dam. In this case the inclusion of a grout curtain reduced total seepage by up to 70%.

For the assumption of one sand layer overlying the rock only a small amount of underseepage (less than 10%) is not intercepted by the blanket drain. Total seepage is reduced by up to 74% under the core and the foundation seepage beyond the downstream toe is reduced by up to 83% by the inclusion of a grout curtain.

Assuming two sand layers within the foundations caused the largest seepage flows. However, more than 90% of this underseepage is intercepted by the blanket drain. Grouting reduced total seepage by up to 83% and reduced seepage by-passing the blanket drain by up to 85%.

In all cases, with an assumed sand layer, grouting reduced seepage flows in the foundation at the downstream toe to small, manageable quantities.

Analyses also studied the effect of relief wells at the downstream toe of the dam. An average reduction of groundwater level to 1 metre below ground level at the relief well line was adequate to collect all seepage under the dam and reverse the flow in sand layers towards the wells.

Exit gradients at the downstream toe of the dam (with no relief wells) reached a maximum of 0.01. Thus there is no risk of erosion or boiling due to seepage at the downstream toe of the dam.

2.6 DECISION TO GROUT

A review of alternative seepage reduction methods included:

(a) grouting
(b) construction of a diaphragm wall under the dam, and
(c) lining the reservoir basin.

A diaphragm wall was not considered feasible due to the expected number of rock 'floaters' in the foundation soil. These would prevent effective wall construction. Lining was rejected on economic grounds.

The construction of a grout curtain through the foundations and upper sections of bedrock was estimated to cost $2 million.

However, due to the extreme sensitivity of the project to environmental issues, the decision was made to proceed.

The nature of the foundation soils required that grouts with the lowest possible viscosity should be used, which, combined with the likely scale of the work, suggested a project unique in Australian grouting experience.

CHAPTER 3 - GROUTING GENERAL

3.1 EARLY HISTORY

It is likely that foundations grouting may have been understood in the Roman times, however, the first documented grout application was by the French Engineer, Charles Berigny (1772-1842) who in 1804 consolidated masonry walls in the port of Dieppe by the injection of a suspension of clay and lime.

The first use of cement grout was by Thomas Hawksley (1807-1893) in 1876 when water bearing fissures in rock were successfully sealed.

The general adoption of 'cementation', as cement grouting was called, was largely due to the work of Albert Francois who developed grouting pumps capable of pumping cement grout at high pressures.

Most early grouting was related to the mining industry but civil engineering use increased dramatically around the turn of the century, particularly related to dam construction.

It was not until 1925 when H J Joosten, a Dutch mining engineer, developed a reliable method of grouting fine grained alluvial materials. The 'two shot' technique involved successive injections of a concentrated solution of sodium silicate and a strong saline solution. Injection was via a pipe fitted with a point and perforated over a length of 600 millimetres of the lower end. The pipe was driven in stages with injection of the silicate taking place at each stage. Injection of the saline solution took place over similar stages during pipe withdrawal.

The need for the two reagents to mix thoroughly beneath the ground limited grout effectiveness to relatively small distances from the injection hole thus close hole spacing was required. This problem led to the development of numerous 'one shot' grouts capable of delayed gelling time with low pre-gel viscosities enabling a considerable penetration distance during injection.

3.2 THE TUBE A MANCHETTE

An important development in soil grouting was the invention of the 'tube a manchette' by E. Ischy in 1933. Prior to the development of the tube a manchette injections were carried out either from the perforated pipes driven into the ground or from boreholes in hard ground.

These methods all required re-drilling of holes for successive injections of grout to the variously permeable zones of the foundations. Often three or more successive injections were required to avoid excessively grouting the more permeable sections.

The tube a manchette, as shown in Fig. 3.1, enabled successive grout applications through a single hole at any selected depth. It consists of a tube of metal or, in recent times, PVC, of between 40 and 60 millimetres diameter with six millimetre diameter holes drilled in the tube wall at, normally, 300 millimetre centres.

The tube wall holes are covered by a tightly fitting rubber sleeve. The tube a manchette is placed in a borehole and the space around the tube is filled with a soft grout of cement and bentonite. Grouting is carried out using a suitably sized pipe, perforated over its bottom length and fitted with packers above and below the area of perforation. This pipe is located within the tube a manchette such that the perforated section is adjacent to one of the tube wall holes.

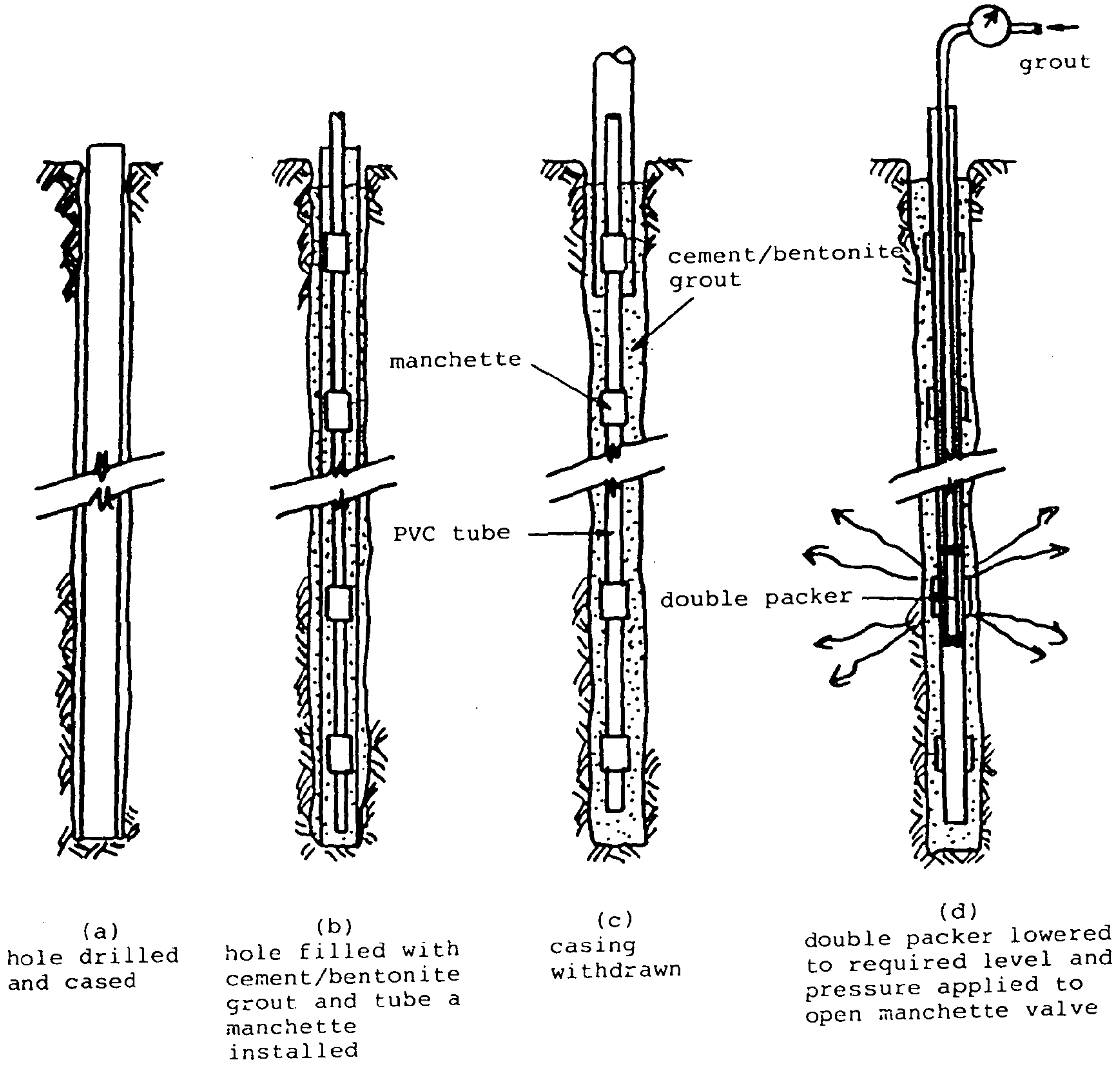


Figure 3.1. Tube a Manchette Grout System

Application of pressure leads to expansion of the rubber sleeve, cracking of the surrounding filling and the injection of grout into the soil.

With the development of the tube a manchette and 'single shot' silicate grouts during the 1930's some major alluvial grouting works were carried out, mainly in France.

The first dam project to involve alluvial grouting was at Genissiat where the coffer dam foundations were underlain by 25 metres of alluvium containing large boulders which prevented the driving of sheet piles. A grout curtain comprising three parallel rows of holes was constructed, the two outer rows being injected with clay grout and the inner row with a sodium silicate grout. The initial work, disrupted by the Second World War, was found to be still effective six years later.

French engineers continued to develop alluvial grouting techniques resulting in a major grouting project, being completed on the foundation of the 130 metre high Serre Poncon Dam in 1951, where over 100 metres depth of sands, gravels and boulders were satisfactorily sealed using the tube a manchette technique.

During recent times rapid advances have been made with chemical grouts and the use of these grouts has become widespread in dam construction, tunnelling and building foundation stabilisation.

3.3 TYPES OF GROUT

Many different grouting materials are currently available, many with distinctive properties to suit specific grouting tasks.

Essentially, grouting involves the filling of voids in soil or rock strata, however, the scale of voids to be filled could range from mine shafts or limestone caverns to microscopic pores in a soil. Similarly the grout may be required to strengthen or stiffen the grouted strata, such as for strengthening of building foundations, or to maintain flexible properties. The grout may be required to seal voids, such as in the grouting of dam foundations.

Most grouts behave as fluids on first mixing, remain fluid during injection, with gellation occurring once the desired penetration has occurred.

Grouts are classified according to their rheological behaviour. The three classifications are:

i) Binghamian Suspensions - suspensions of cement or clay.

ii) Colloidal Solutions - silicates, lignochromes, bitumen emulsions and organic colloids.

iii) Pure Solutions or Resins - organic monomers in water solution.

Pure solution grouts are also known as Newtonian grouts. These are liquids without rigidity, whose viscosity is independent on their flow velocity.

Binghamian grouts have an inherent rigidity and a viscosity proportional to flow velocity.

Colliodal solution grouts behave initially as Newtonian fluids, but progressively develop Binghamian fluid properties as gelling occurs.

The rheology of these grouts is discussed in detail in Chapter 4.

The various materials available are discussed in detail below.

3.3.1 Binghamian Suspensions

a) Cement

By far the most common grouting material is cement, injected in a suspension form. Normally a Type A Portland cement or a high early strength Type B cement is used to take advantage of additional particle fineness.

The cement is mixed with water in a high speed mixer to ensure the maximum dispersion of the cement particles. Water cement ratios are usually between 3 and 0.5 to one by volume.

Additives are often used to vary the grout properties for various reasons. For example fillers of clay, pozzolan or fine sand are common additives. Clay, usually bentonite, has the ability of "stabilising" the cement suspension, reducing the tendency for bleeding or settling of the cement particles thus increasing the penetrability of the grout in fine fissures. Methocell has similar properties. Pozzolans such as flyash can be used in conjunction with cement to produce a low cost grout. Sand can similarly form an economical filler where high solids, low water grout is required to seal large voids.

Other typical concrete admixtures can be used in grout for accelerating, retarding, air entrainment and shrinkage reduction.

The recent development of an "ultra fine" cement grout has been reported (Massao 1982) which is claimed capable of penetrating sands with coefficients of permeability of 10^{-5} to 10^{-6}m/sec.

Cement grouts are commonly used for sealing coarse soils or fractured rock in dam foundations. These grouts are also used for consolidation grouting of building foundations and all general grouting applications.

b) Clays

Clays can be used to form grout due to their small particle size and ability to form gels. A wide range of economic grouts can be formulated using clay, either alone or in combination with other materials, for grouting coarse soils through to fine sands.

Only clay materials able to absorb water and form gels make useful grouts. Sodium and calcium montmorillonites have exceptional properties in this regard. Bentonite is a naturally occurring clay, rich in these minerals and is very commonly used in clay grouting. Bentonite in dry powder form is available in bulk or bagged like cement.

The small size of clay particles enables effective penetration of soils with permeabilities of 10^{-4}m/sec and lower. However, pure clay grouts are unable to resist high hydraulic gradients and are unsuitable for use in variable permeability soils.

The thixotropic properties of clay suspensions can be a problem in clay grouting and often dispersive agents are added to overcome this effect.

Clay grouts are commonly used in dam foundations where voids are too fine for effective penetration of cement grouts.

3.3.2 Colliodal Solutions

a) <u>Sodium Silicate Grouts</u>

Sodium silicate, $nSiO_2.Na_2O$ is commercially available in an aqueous (colloidal) solution. When this solution and an appropriate concentrated salt solution are mixed, a gel is rapidly formed. This is the basis of the Joosten process, the earliest recorded example of "chemical grouting" which required the two reagents to be injected separately in what was known as a 'two shot' process.

The use of more dilute solutions of sodium silicate and different salts enabled a delayed gel or 'one shot' grout to be developed.

Early one shot sodium silicate grouts such as the sodium silicate/sodium bicarbonate mixture, had low viscosity but low strength and limited gel life.

An improved mixture used formamide as a reactant until this was found to be possibly carcinogenic and its use declined.

Currently organic reactants are being investigated.

The strength of a silicate grouted soil is directly related to the silicate content. The viscosity is also proportional to the concentration of the grout solution. Thus where soil strength is a design criteria grout viscosities around 10 centipoise must be used. Where strength is not important viscosities of 3 to 4 centipoise are possible.

The long term performances of sodium silicate is questionable, particularly in alkaline conditions. Two undesirable characteristics have been observed.

The first is the phenomena of synerisis which is the loss of water and shrinkage of a newly formed gel. In grouted soils this can lead to the development of a residual permeability. The phenomenon is less pronounced in finer soils.

The second characteristic is the tendency for the acidic gel to be dissolved by alkaline conditions resulting from external groundwater sources or from isolated areas of excessive soda content resulting from low reactant concentrations and long setting times. It is usually considered unlikely that high internal soda contents could cause large scale gel breakdown unless field error or improper mix design was involved.

A major advantage of sodium silicate grouts is that they are considered non toxic and free from health or environmental hazard. However, some of the reactants may not be so inocuous.

Sodium silicate grouts are economical and are perhaps the most widely used of the 'chemical' grouts.

They are commonly used for consolidation of loose sands in building and tunnelling works and have been extensively used in sealing dam foundations in sandy materials.

b) Lignosulfonate Grouts

Lignosulfonate grouts are produced from a by-product of the wood processing industries. They consist of a mixture of lignosulphonate with a chromium compound normally sodium dichromate. In an acid environment the chromium ion oxidises the lignosulphonate, forming a gel.

The normal range of grout viscosities is 3 to 8 centipoise for a 200 to 600 gram per litre mix. Viscosity increases during the pre-gel period.

The gels are considered permanent in continually wet conditions but they deteriorate in wet/dry or freeze/thaw conditions.

The grouts are very economical, although the dichromate salt is highly toxic. Whilst the resulting gel is non toxic, the reaction is often incomplete and toxic material can be leached from the grout.

These grouts are not commonly used.

3.3.3 Pure Solutions

a) <u>Acrylamide Grouts</u>

Acrylamide grouts were the first of the pure solution chemical grouts, the most well known being AM-9, introduced in 1951. The grouts are mixtures of organic monomers whose setting times can be accurately controlled by catalyst percentage. Their viscosity and density are close to water with viscosity remaining virtually constant prior to sudden gelling.

A typical grout mixture would be 95% acrylamide, polymerised into long molecular chains, together with 5% of cross linking agent to bind the chains together. The stiffness or strength of the gel can be varied by varying the proportion of the monomers.

Typical grouts contain 8% to 10% solids which form a randomly linked structure mechanically trapping the water molecules. Up to 10% of the trapped water can escape in low moisture conditions causing shrinkage of the gel. Similarly the gel can re-absorb water and expand to its original volume in wet conditions. Shrinkage cracks will seal up but not chemically rebond.

Various catalysts, accelerators or inhibitors are used with acrylamides, some of which are considered to pose health hazards.

Gellation is exothermic, the temperature rise contributing to the speed of gellation. Gel time can be affected by groundwater conditions, with sodium chloride, for example, speeding the reaction.

Acrylamide grouts are considered permanent with the gel being unaffected by exposure to all but very strong acids and bases.

A major drawback to the use of acrylamide grouts is their neurotoxicity.

The manufacture of AM-9 was discontinued in 1978, although several other brands are still marketed including Rocagil BT, Nitto SS and Terragel.

It is currently claimed that whilst the acrylamide in powder or solution form is neurotoxic, the gel is non toxic. The degree of health hazard involved in using the grouts is well known and can be overcome by simple handling procedures.

A new grout Injectite - 80, based on poly-acrylamides was introduced in 1980. It uses a completely different catalyst system to the mono acrylamides and avoids the toxicity problems, albeit with a sacrifice of viscosity.

Acrylamide grouts have low viscosity and have in the past been used extensively in sealing dam foundations. Their reputation for toxicity has, however, led to their current use being greatly curtailed.

b) Phenoplasts

Phenoplasts result from the reaction of a phenol on an aldehyde. The most common grout mixture involves resorcinol and formaldehyde with a catalyst, usually sodium hydroxide, to control pH.

Setting time is directly related to solution concentration and temperature.

Initial viscosity ranges from 1.5 to 3 centipoise and, as with acrylamides, viscosity is essentially constant until gellation starts.

The gel is considered permanent except when exposed to alternating wet/dry conditions.

The grout components all pose health hazards. Resorcinal is toxic and caustic, although not to the extent of other phenols. Formaldehyde can cause chronic respiratory ailments and sodium hydoxide is highly caustic.

Properly proportioned, however, the gel is inert.

Several commercial grouts based on Phenoplasts are available. These include:

Rocagil - Viscosity 5 to 10 centipoise.

Geoseal - Viscosity 2 to 10 centipoise.

Terranier - A phenol base with dichromate salt.

These grouts are easy to use and have in recent years been used to seal dam foundations in low to moderate permeability soils where acrylamides are considered unsuitable due to their toxicity.

c) <u>Aminoplast Grouts</u>

Aminoplast grouts are based predominantly on urea and formaldehyde. They require an acidic environment for gelling and cannot be used where groundwater conditions have pH greater than 7.

Aminoplast grouts have low viscosities, but the reaction of the extremely low viscosity urea solutions is rapid and difficult to control. It is thus normal to use a partially reacted prepolymer which can be more predictable although viscosity is normally between 10 and 20 centipoise.

Little data is available on these grouts, however, it is claimed that they are permanent except in cyclic wet/dry or freeze/thaw conditions.

The grouts are toxic and corrosive prior to gelling due to the formaldehyde and acid catalyst.

Commercially available grouts include a Rocagil version, specially developed for coal mine use, together with Herculon, Diarock and Cyanaloc 62.

These high strength grouts are commonly used for consolidation grouting.

d) <u>Water Reactive Materials</u>

Materials which gel or polymerise on contact with water have been investigated for use as grouts. These include several 'foaming' grouts.

Polyurethanes in particular appear to have significant potential for specialised applications for sealing large voids including backfilling behind tunnel linings.

Two commercially available products are the TACSS system, which offers grouts with viscosities ranging from 22 to 300 centipoise, and CR250, initially used for sealing sewer leaks but now also considered for soil grouting.

e) Acrylate Polymer Grout

A new grout system introduced in 1980 to replace the discontinued Acrylamide grout AM-9 was AC-400, based on acrylate monomers polymerised with an oxidant/reduction catalyst.

The grout is claimed to have virtually identical properties to AM-9 but only 1% of the toxic exposure.

Viscosity is 2 centipoise which would make the grout particularly attractive for sealing low permeability soils.

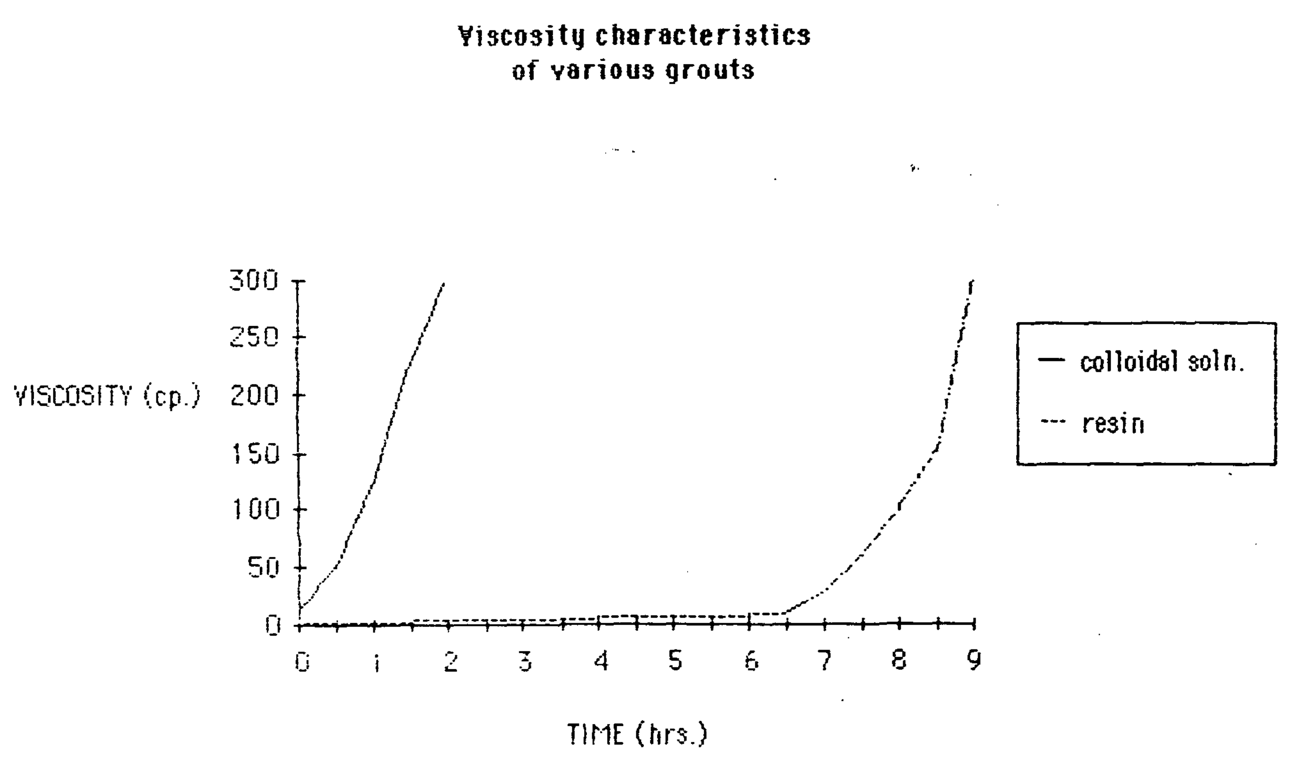


Table 3.1 summarises the chemical grouts available and their properties.

TABLE 3.1
CHEMICAL GROUTS

GROUT TYPE	CORROSIVITY OR TOXICITY	VISCOSITY	STRENGTH
SILICATES			
Joosten Process	Low	High	High
Siroc	Medium	Medium	Medium/High
Silicate/ Bicarbonate	Low	Medium	Low
LIGNOSULPHATES			
Terra Firma	High	Medium	Low
Blox-all	High	Medium	Low
PHENOPLASTS			
Terranier	Medium	Medium	Low
Geoseal	Medium	Low/Medium	Low
Rocagil	Medium	Medium	Low
AMINOPLASTS			
Herculon	Medium	Medium	High
Cyanaloc	Medium	Medium	High
ACRYLAMIDES			
AV-100	High	Low	Low
Rocagil BT	High	Low	Low
Nitto SS	High	Low	Low
Terragel	High	Low	Low
POLY ACRYLAMIDES			
Injectite 80	Low	High	Low
ACRYLATE			
AC-400	Low	Low	Low
POLYURETHANE			
CR-250	High	High	High
CR-260	High	Medium	High
TACSS	High	High	High

CHAPTER 4 - GROUTABILITY OF SOILS

4.1 INTRODUCTION

The major consideration in any proposed grouting operation is the nature of the foundation material to be grouted, particularly when the foundation material is a soil.

A successful project will achieve the combination of an appropriate grout and an appropriate injection technique. Caron (1980) describes three methods by which grout may penetrate a soil. These are by:

i) impregnation: where the grout penetrates existing fissures or voids,

ii) fracturing: where the grout penetrates voids created by the physical process of grouting,

iii) a combination of fracture and impregnation.

Penetration by fracturing alone, assuming negligible penetration of pre-existing voids or features, is a special and unusual form of grouting and of limited application where the aim of grouting is for control of permeability. Intentional applications of this form of grouting are for consolidation of weak, saturated soils, for lifting of structures or for creation of underground structures from grout.

Penetration by impregnation is the desired method of grouting to achieve a reduction in permeability. It relies on the selection of grouts with viscosity and particle size sufficiently low to flow, under moderate pressures, into and fill the soil voids or features.

The third method discussed by Caron, involving both fracture and impregnation is probably the most common method of grouting in soils, although, the degree that fracturing is involved may not always be recognised.

As the grout pressure is increased the rate of flow into the soil structure is also increased. However, eventually the pressures in the soil can lead to an exceeding of the bond strength between soil particles and fracturing will take place. The effect of fracturing is an increase in the effective area of grout application and a consequent increase in the rate of grout acceptance by impregnation.

The capacity of a soil to accept a grout by either method is termed "groutability".

As will be discussed in this Chapter, groutability is a function of many variables, but the primary influences are the permeability of the soil and the rheology of the grout.

4.2 PERMEABILITY

A sample of soil consists of an assemblage of many individual soil particles with air and/or liquid filling the voids between the particles.

In general, all voids in a soil are connected to neighbouring voids. Electron photomicrographs have shown this to be true even in the finest grained soils. Fluids can thus pass through soils.

Experimental work carried out by H. Darcy in the 1850's derived the well known relationship for flow of water in a soil.

$$\frac{Q}{A} = v = ki \qquad 4.1$$

Subsequent work has shown Darcy's law to apply to most types of fluid flow in soils.

By consideration of Poiseuilles Law relating to the distribution of velocity in capillary tubes, Taylor (1948) was able to deduce that:

$$k = d_s^2 \cdot \frac{\gamma}{\mu} \cdot \frac{e^3}{1+e} \cdot C \qquad 4.2$$

Where
- d_s is an effective particle diameter (m)
- γ is the density of the permeant (N/m^3)
- μ is the viscosity of the permeant (Pa.s = 10^3cP)
- e is the void ratio
- C is a factor related to the shape of the soil particles

Equation 4.2 demonstrates the major influences on the permeability of any soil, namely:

a) The soil properties - grading
- particle size
- void ratio
- soil particle shape & orientation

and

b) The permeant properties - density
- viscosity

Lambe, et al (1969) describes a similar relationship known as the Kozeny Carman equation:

$$k = \frac{1}{k_o S^2} \cdot \frac{\gamma}{\mu} \cdot \frac{e^3}{1+e} \qquad 4.3$$

Where k_o is a factor depending on pore shape and ratio of the length of the actual flow path to soil bed thickness and S is the surface area of the particle with diameter d_s.

Many researchers have investigated this relationship and found that it is insufficient for a full description of the fluid behaviour in a soil. Nevertheless, it adequately describes the principles involved and can be used to develop an understanding of many aspects of grouting.

4.3 RHEOLOGY

For liquid and gases in laminar flow conditions the velocity gradient is proportional to the applied shear stress, the inverse of the coefficient of proportionality being defined as the dynamic viscosity of the fluid.

Hence: $$v = \frac{\tau}{\mu} \qquad 4.4$$

Where τ is the applied shear stress
μ is the viscosity
and v is the velocity of flow.

Fluids for which this relationship is true are known as Newtonian fluids.

Some liquids however, as shown by Bingham, do not behave in accordance with this relationship. Such liquids, particularly suspensions, do not start flowing until a critical stress, defined as the yield value, is reached. Flow velocity at stresses above this yield value becomes proportional to the excess stress over and above the yield value with the constant of proportionality being defined as the dynamic or plastic viscosity.

This relation can be written:

$$\tau - \tau_f = \mu_{pl} \frac{d\varepsilon_s}{dt} \qquad 4.5$$

Where τ_f is the Bingham yield stress
μ_{pl} is the plastic viscosity
$\frac{d\varepsilon_s}{dt}$ is the rate of change of shear strain or velocity

Mayer (1963) suggested that the existence of the yield value is related to electrical forces of attraction between the particles scattered throughout a suspension.

In some instances the yield value varies according to the previous state of activation of the suspension, being higher when the suspension is allowed to stay at rest and diminishing on agitation. Suspensions with this property are called thixotropic.

Viscosity is measured in the laboratory using a viscometer, a device with coaxially rotating cylinders, the shear between which can be measured.

However, more practical measurement techniques have been developed for use in the field.

The most common of these is the Marsh Cone which comprises a calibrated funnel. Viscosity has been shown to be proportional to the flow time of a certain fixed volume of fluid from the cone.

A further rheological property of importance to grouting is the "stability" of a suspension, a stable suspension being one with little tendency to separate into solid and liquid prior to completion of the grouting operation and grout gelation. This property is very dependent on the grain size of the solid particles and their surface area. Cement grout, for example, is relatively unstable. The stability of a cement grout can be improved by using fine cement, violent agitation to break down particles or by addition of a material such as bentonite which forms a highly stable suspension.

4.4 GROUT PENETRATION BY IMPREGNATION

Apart from obvious relationships with viscosity and permeability, grout penetration by impregnation has been found to be dependent on numerous other factors. Mayer (1963) described several factors influencing the penetration of, particularly, suspension grouts. These include:

a) Grain size - particles tend to form a natural filter structure despite having smaller size than the cavities to be penetrated.

b) Grain shape - smooth, rounded particles tend to penetrate more readily.

c) Chemical properties - adhesion or reaction of the grout to the rock or soil mass can be significant.

d) Grout techniques - high pressures can temporarily enlarge openings to enable penetration of grouts that would appear incompatible with the original nature of the voids.

Scott (1963) demonstrates that the nature of grout penetration is significantly different for pure solutions, or Newtonian grouts, and for suspension, or Binghamian grouts.

For Newtonian grouts, the grout flow rate is directly proportional to the injection head and according to Scott can be described by the equation:

$$\frac{q}{h_i} = 4\pi \, a_1 \, k_g \qquad 4.6$$

where q is the grout flow rate

h_i is the injection head

a_1 is the effective spherical radius of the source, and

k_g is the coefficient of permeability of the soil to the grout

The radius a_1 is approximately by:

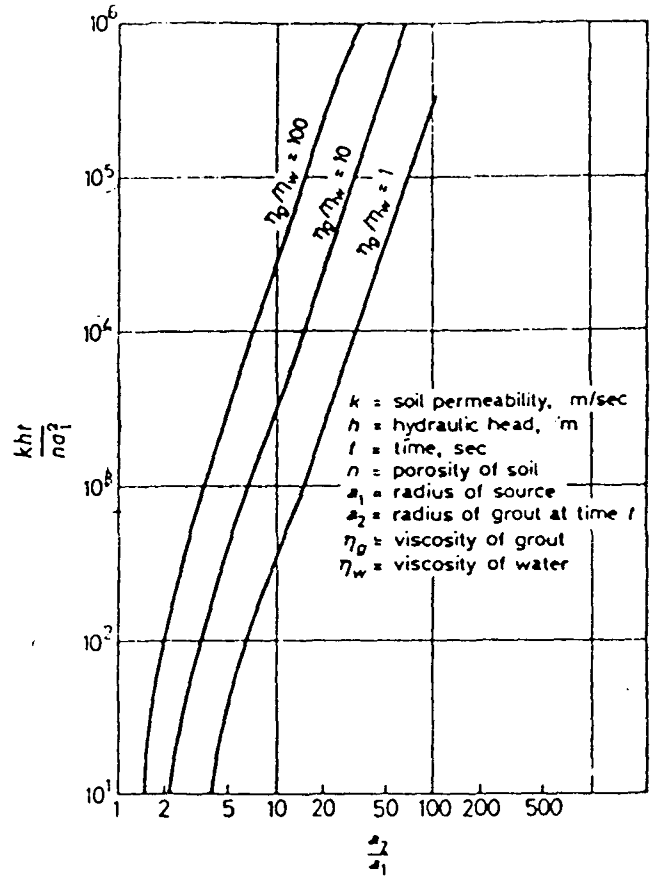
$$a_1 = \frac{\ell}{\log \, (L_i+\ell) \, (L_i-\ell)} \qquad 4.7$$

Where $\ell = (L^2 - 4rh^2)^{1/2}$

For L_i = injection length

r_h = hole radius

The equation is slightly modified for the case where grout is flowing into a water saturated medium and the resulting relationships are shown in Figure 4.1 (after Raffle and Greenwood (1961)).



Penetration time for various viscosity ratios η_g/η_w

Figure 4.1.
(after Raffle & Greenwood)

For Binghamian or Non-Newtonian grouts an additional pressure gradient must be applied to overcome the internal shear stresses during injection. Raffle and Greenwood (1961) deduced that the additional pressure gradient required was:

$$\frac{dh}{dr} = \frac{2\,\tau_f}{R_p} \qquad 4.8$$

where τ_f is the Binghamian yield stress and R_p is the effective soil pore radius.

They were able to assess the value of R_p from the Kozeny relation (see equation 4.3)

Table 4.1 summarises the hydraulic gradients needed to maintain flow of grouts with various yield values in soils of various water permeability. Figure 4.1 can be used to assess the penetration rate of the grout for pressure components above these values

Table 4.1

Gradient to maintain flow in non-Newtonian grouts

Soil permeability m/sec	Yield value T_f Pa	Minimum hydraulic gradient
10^{-2}	1	1.2
	10	12
	100	120
	1,000	1,200
10^{-3}	1	4
	10	40
	100	400
	1,000	-
10^{-4}	1	12
	10	120
	100	1,200
	1,000	-
10^{-5}	1	40
	10	400
	100	4,000
	1,000	-
10^{-6}	1	120
	10	1,200
	100	-
	1,000	-

To demonstrate the differences between grouts it is useful to consider two quite different grouts, namely:

a) a pure solution grout with viscosity 4 cp (4×10^{3} Pa.s), and

b) a 5% solution of bentonite,

both being injected into a soil with permeability 10^{-5}m/sec and porosity of 0.35 at a depth of 10 metres with a desired penetration distance of 1.5 metres.

The bore hole is assumed 75 millimetres diameter and the length of injection 0.3 metres.

From equation 4.7 the effective radius of the source is

$a_1 = 0.16$m

Considering the pure solution grout using figure 4.1 for $a_2 = 1.5$m, $\frac{\mu_g}{\mu_w} = 4$, $n = 0.35$ the relationship $ht = 7.6 \times 10^5$m sec is derived.

For grout head h restricted to say 25 metres to prevent excessive fracture or uplift of the soil it can be seen that grout penetration would be complete in 3000 seconds, or 50 minutes.

Hence grouting is complete in less than one hour and appears to be a practical operation.

For the bentonite grout typical values for viscosity and for shear strength would be 10 centipoise (10^2 Pa.s) and 5 kPa respectively (after Raffles and Greenwood).

From figure 4.1, for $\frac{\mu_g}{\mu_w} = 10$

The relationship $ht = 2.7 \times 10^6$m sec is derived

For completion of grouting within one hour a head of 750 metres would be required. In addition, from Table 4.1 with $\tau_f = 5$ an hydraulic gradient of 200 would be required to overcome the bentonite grout's internal shear stress. This would represent an additional 350 metres head at the hole. The total head required represents a pressure of 11,000 kPa, clearly impractical.

A more realistic sequence of events when attempting to grout this soil with bentonite grout would be a slow penetration to 200 to 300 millimetres at which point the pressure required to maintain any sort of flow would lead to fracture of the ground and either surface leakage or ground heave.

For the same bentonite grout in soil with a permeability of 5×10^{-3}m sec it can be shown that the desired penetration can be achieved in approximately 15 minutes.

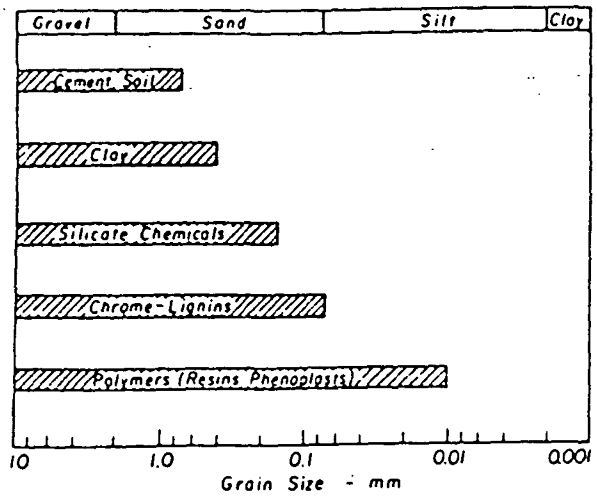
These examples clearly indicate the sensitivity of a grouting operation to the physical properties of the grout and the need for careful consideration in the design of an economical and effective grout system, particularly in soils with layers or seams of varying permeabilities.

Various authors have proposed guidelines for assessing the range of soils which are practically groutable for the various grouts.

Fig. 4.2 presents examples of these guidelines (ex Klavetz (1958), Caron et al (Winterhorn and Fang), Mitchell (1970)).

Often a number of different grouts must be injected in sequence to avoid excessive use of an expensive, low viscosity grout or ineffective sealing of less permeable features by a cheap grout. The injection of grouts in sequence is common practice with the tube a manchette technique discussed in Chapter 3.

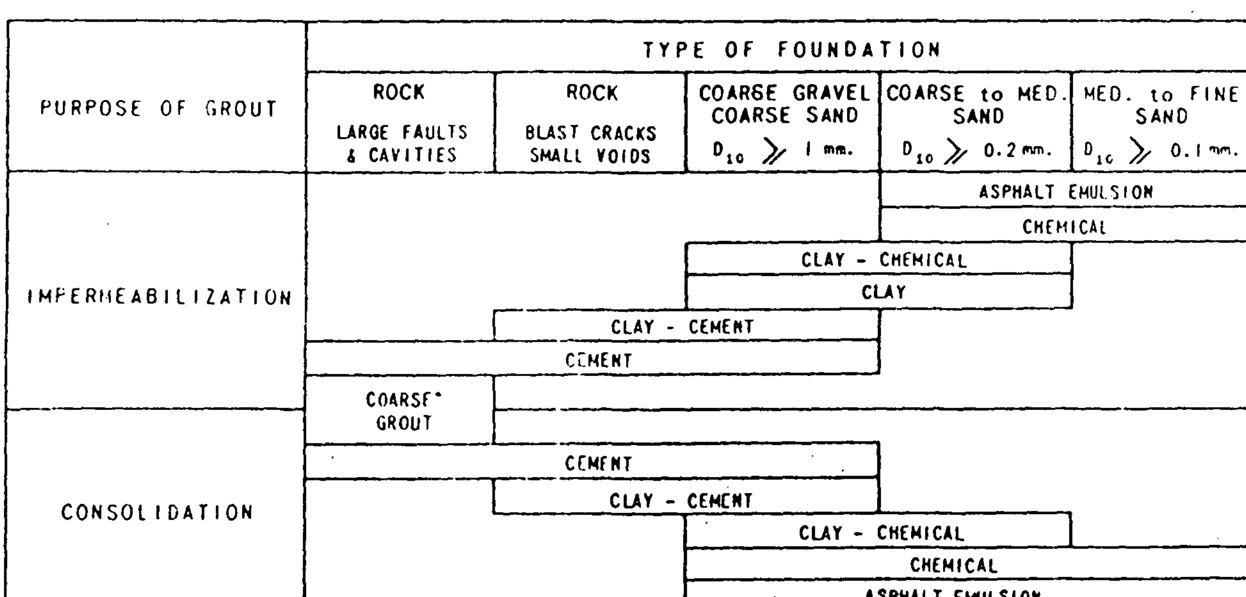
Fig. 4.3 shows diagrammatically, the effect of using a multiple grout system for sealing of a soil with lenses of permeability varying from 10^{-2} m/sec to 10^{5} m/sec.



a) after Mitchell (1970)

Type of Soils	*Coarse Sands and Gravels*	*Medium to Fine Sands*	*Silty or Clayey Sands, Silts*
Soil Characteristics			
Grain Diameter	$d_{10} > 0.5$ mm	$0.02 < d_{10} < 0.5$ mm	$d_{10} < 0.02$ mm
Specific Surface	$S < 100$ cm^{-1}	100 cm^{-1} $< S <$ 1000 cm^{-1}	$S > 1000$ cm^{-1}
Permeability	$K > 10^{-3}$ m/sec	$10^{-3} > K > 10^{-5}$ m/sec	$K < 10^{-5}$ m/sec
Series of Mix	Bingham suspensions	Colloid solutions (Gels)	Pure solutions (Resins)
Consolidation Grouting	Cement ($K > 10^{-2}$ m/sec)	Hard silica gels: Double shot: Joosten (for $K > 10^{-4}$ m/sec)	Aminoplastic
	Aerated mix	Single shot: Carongel, Glyoxal, Siroc	Phenoplastic
Impermeability Grouting	Aerated mix	Bentonite gel	Acrylamide
	Bentonite gel	Lignochromate	Aminoplastic
	Clay gel	Light carongel	Phenoplastic
	Clay cement	Soft silicagel	
		Vulcanizable oils	
		Others (terranier)	

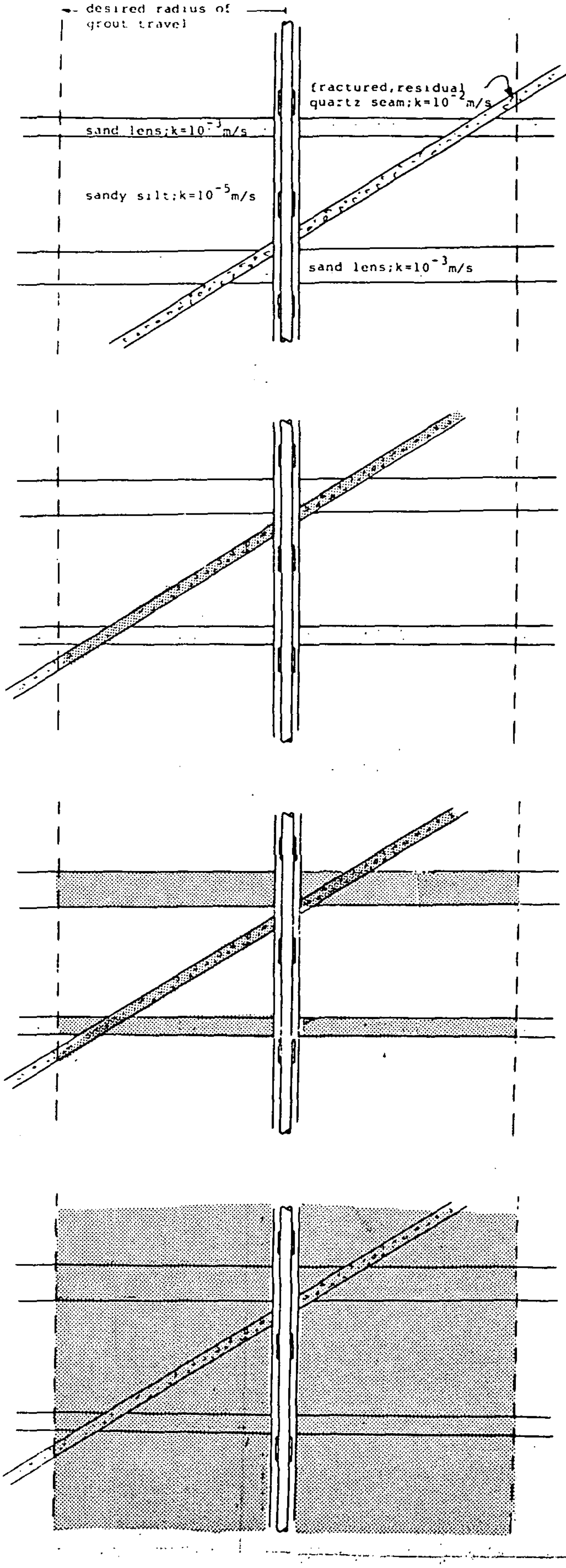
b) after Caron



* saw-dust, sand-cement-clay, etc.

c) after Kravetz (1958)

Figure 4.2. Limits of Groutability



hypothetical soil profile

Phase 1 grouting
- cement/bentonite

Phase 2 grouting
- clay or silicate

Phase 3 grouting
- pure solution

Figure 4.3. Multiple Grout System

The economic design of a grout programme would involve detailed consideration of the costs of the grout and the grout injection.

Several examples of actual multi-grout projects are discussed in Chapter 6.

Other considerations relating to grout impregnation include the effects of grout front mixing and sinking of grout masses prior to gelling. These are discussed by Scott (1963).

Where the grout viscosity is close to or lower than that of the displaced fluid (usually water) the grout front is unstable and the grout front tends to develop sinuosities or fingers. This can lead to dilution of the grout and result in ineffective grouting.

Similarly grouts denser than the displaced fluid tend to sink under gravitational forces and the front can become unstable.

Saffman and Taylor (1958) showed that for grouts denser and more viscous than water the grout front remains stable during pumping, provided that

$$R_1 \frac{h \mu_g a_1}{\mu_g (a_2 - a_1) + \mu_w a_1 R_2} > \frac{\gamma_g - \gamma_w}{\gamma_w} \qquad 4.9$$

for spherical flow, where a_1 is radius of the source and a_2 the distance of the grout front from the source.

Referring to the examples of Section 4.4, we find that for the pure solution grout, with a density 1.2 times that of water a minimum head of 2.6 metres is required to maintain stability to 1.5m radius.

For the bentonite grout with a density of 2 times that of water a minimum head of 25m is required to maintain stability.

Scott suggests that mixing on the grout front has not been a practical problem for fine grained soils even for low viscosity chemical grouts. It is likely that surface tension and other effects become significant in these soils and the mathematical predictions of equation 4.9 are not accurate.

4.5 GROUT PENETRATION BY SOIL FRACTURING

The applied pressure at a grout hole is dissipated in mobilisation of the grout, in the case of a Binghamian fluid, and in the forcing of a flow of grout into the soil voids.

The rate of grout flow into the soil is proportional to the surface area of the injection hole, the permeability of the soil, and the hydraulic gradient across the grouted zone. The latter is proportional to the pressures applied at the grout hole.

As the pressure at the grout hole is increased, a point is reached where tensile stresses are induced which exceed the soil strength and fracturing of the soil occurs. This phenomenon, known as 'hydraulic fracturing' or 'hydrofracture' is dealt with fully in Chapter 5.

The immediate effect of a fracture is to increase the effective area of the grout hole, thus increasing the potential grout flow rate for a particular grout hole pressure.

For a pumped grout system the occurrence of a fracture would lead to a sudden drop in pressure and consequently a reduction in the potential for the initiation of additional fractures or for extension of the initial fracture.

Thus, in situations where grout pressures are only moderately higher than fracture pressures, and adjustments to grout valves and pumps are carried out slowly, hydraulic fracture can be a relatively controlled operation. The increase in effective grout hole area and consequently more rapid grout take can have significant financial implications on larger grouting projects. However, in situations where grout pressures are significantly higher than fracture pressures, and where equipment adjustments are relatively rapid, fracturing can be extensive and uncontrolled. In low permeability soils, for example, the rate of flow of grout into the soil by impregnation is very slow. Conventional grouting equipment however, comprises positive displacement pumps which can develop extremely high pressures at low flow rates. Pressure to the grout holes is controlled by a two way valve system off a return line as shown in Fig. 4.4. Properly operated, this system gives moderately good control over pressures at the hole. However, when an injection system is aiming at a fixed injection volume per stage or a specific grout injection rate, then small scale adjustments can lead to large pressure increments and uncontrolled fracturing.

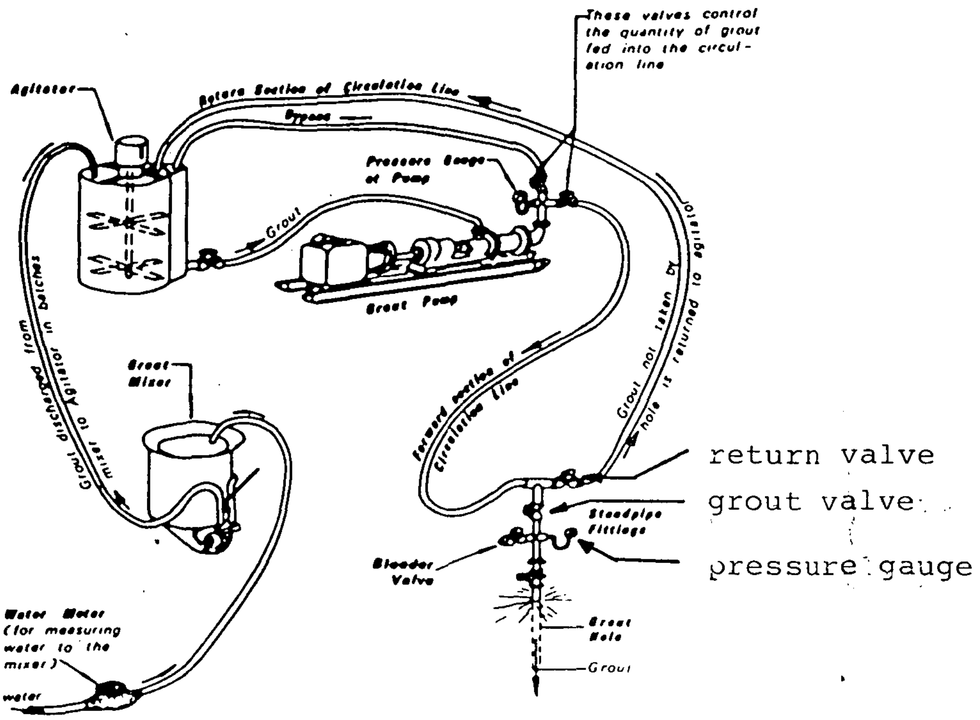


Figure 4.4. Typical Grouting Equipment
(after Houlsby)

Type of ground / Type of grouting method	Coherent fissured soils: Large and medium fissures $K > 5.10^{-7}$ m/sec	Coherent fissured soils: Very fine fissures $K < 5.10^{-7}$ m/sec	Loose soils: Coarse and medium $K > 10^{-3}$ m/sec	Loose soils: Fine $K < 10^{-3}$ m/sec
by fracturing			Cement-based grout	Cement-based grout
by impregnation	Cement-based grout	Chemical grout	Cement-based grout	Chemical grout
by fracturing-impregnation				Chemical grout

Cement-based grout

Chemical grout

No case for grouting

Figure 4.6. Grout Method vs. Soil Type
(after Caron)

Whilst it may be argued that the extending fracturing may intercept and enable sealing of isolated lenses of more permeable material, it is more likely that the randomly generated fractures could extend well past the intended grout curtain limits with the bulk of injected grout being used in filling the self propogated fissure and having negligible effect in reducing overall permeability.

Fig. 4.5 indicates the predicted effects of controlled and uncontrolled fracturing on grout impregnation.

Experience at the Worsley Alumina Refinery confirmed a suspicion that fracturing of the low permeability clayey-silt soil was inappropriate as will be discussed in detail in Chapter 7.

Vaughan (1963) reported a similar suspicion in quite different material at Balderhead Dam.

Other published experience is not available but Caron (1982) suggests that fracturing is appropriate only in 'loose' (sic) soils. Fig. 4.6 reproduces a table from Caron, which suggests the appropriate grout penetration method for various situations.

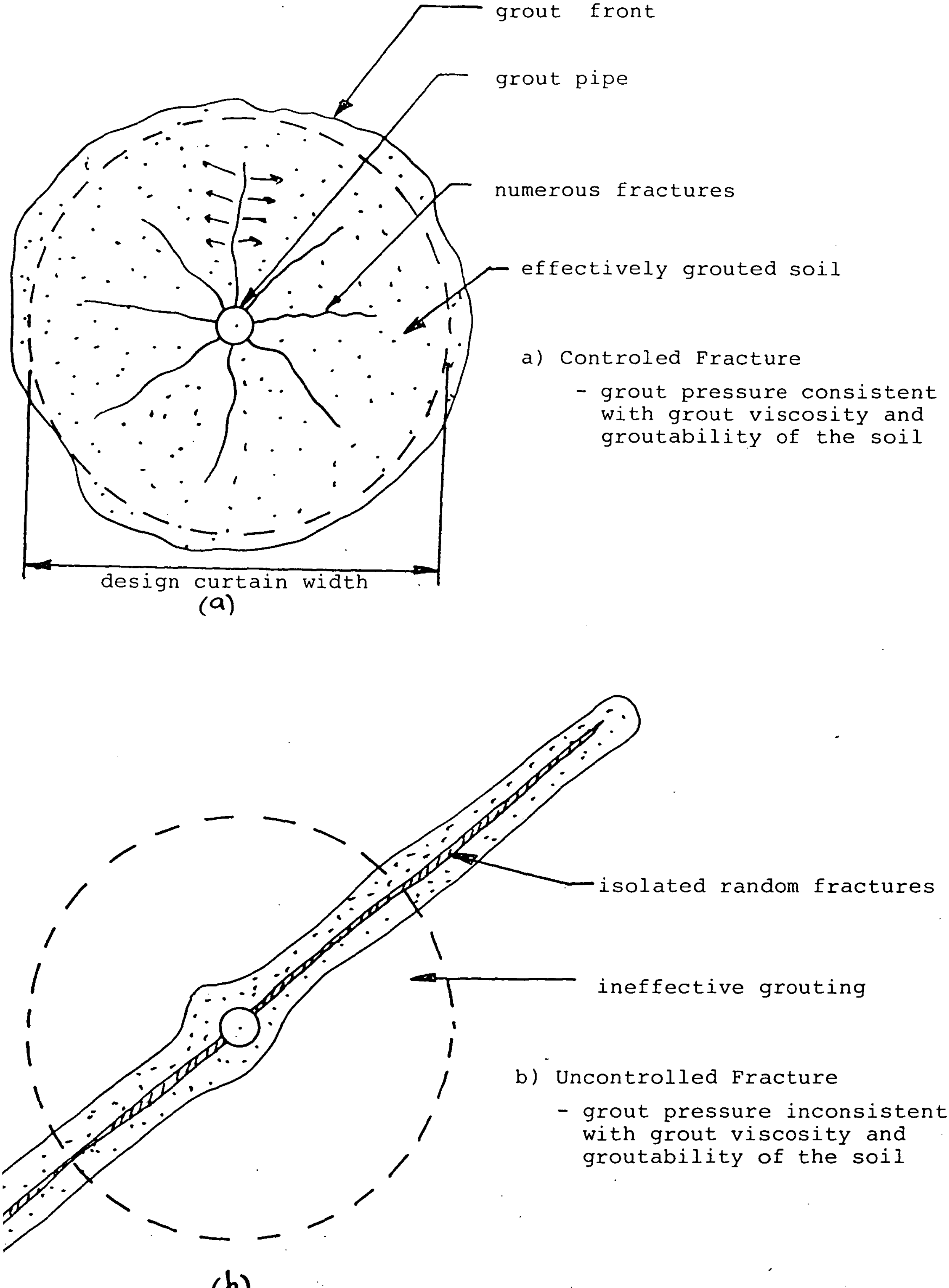


Figure 4.5. Effects of Hydraulic Fracture

CHAPTER 5 - HYDRAULIC FRACTURE

5.1 INTRODUCTION

When hydraulic forces in a soil or rock medium exceed the surrounding ground pressures, induced tensile stresses can lead to fracture of the ground. This phenomenon is known as hydraulic fracture or hydrofracture.

The concept has been well known to drillers in the oil industry since the 1930's. It has been used to improve the productivity of oil wells by fracturing of the source rock and creation of flow paths by injection of large volumes of water or acid. (Massarach 1978).

More recently the process has been suspected as being a potential cause for failure of earth dams (Jaworski et al 1981, Vaughan 1971) and has become recognised as an important consideration in the pressure grouting of foundations (Massarach 1978, Wong and Farmer 1973). The latter aspect is investigated in detail in this Chapter.

5.2 OBSERVED NATURE OF FRACTURES

Hydraulic fracturing has been studied in the laboratory (Jaworski et al 1981, Bjerrum et al, 1972, Enever et al 1976), in the field by others (Bjerrum et al, 1977, Wilkes, 1974) and by the author, at the Worsley Alumina Refinery.

Jaworski carried out a laboratory test on samples taken from the core of Teton Dam.. The samples were compacted to 7.5 inch cubes and placed in a cubical stress apparatus. Stresses applied were such as to create a K_o of 0.5.

Water was injected through a 4.88mm diameter hole in the sample.

A typical test result is shown in figure 5.1.

Single fractures developed in these samples and results indicated that fracturing would always commence in a weakness in the soil structure.

Several tests were carried out to evaluate the effect of test duration and also determine the effect of a pre-existing fracture. It was found that much higher pressures could be sustained without fracturing if pressures were increased slowly (Fig. 5.2). It was also shown that refracturing occurred at a significantly lower pressure than initial fracturing although some 'healing' may occur with time and increased soil pressures (Fig. 5.3).

Fractures were all essentially vertical, being perpendicular to the minor stress axis.

Bjerrum et al (1972) carried out a series of laboratory experiments involving small scale piezometers in a glass walled tank filled with soft silty clay. A rapid increase in flow rate was noted following piezometer pressure increase and coincided with the observation of a crack through the soil. Once again a single crack occurred, only in this case it was approximately horizontal, attributed to placement effects of the clay in the apparatus.

Enever et al (1976) carried out similar tests in a triaxial apparatus and observed single vertical cracks. It was noted that "it was not possible to obtain a fracture which did not propagate to the boundary instantaneously".

In the field, Bjerrum (1972) was involved with permeability tests at several earth embankments including Hyttejuvet Dam in Norway.

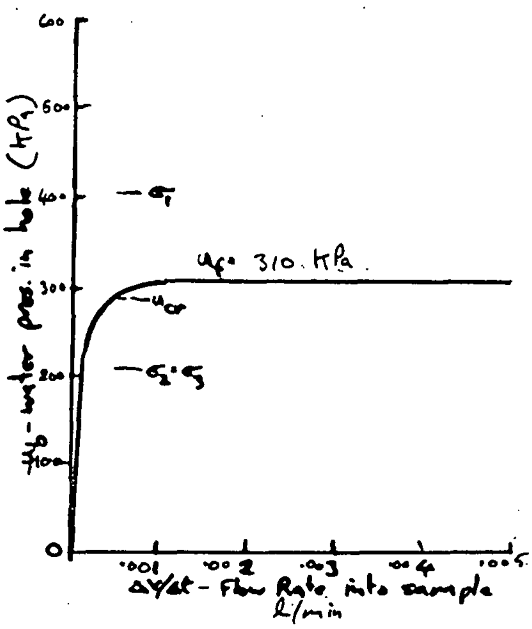


Figure 5.1. Typical hydraulic fracture test result
(Jaworski 1981)

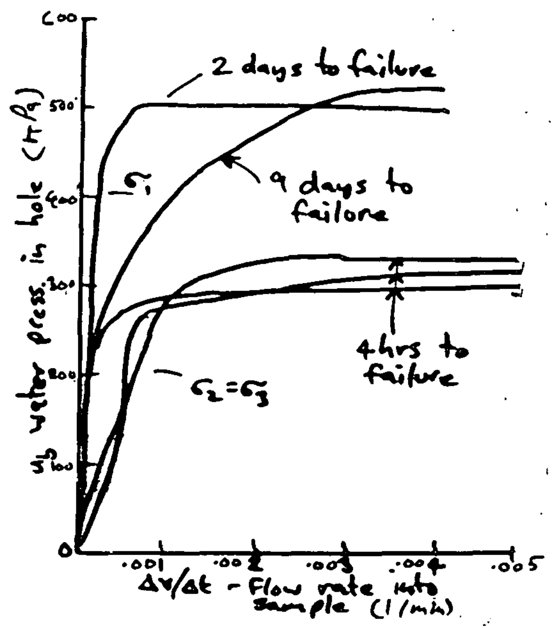


Figure 5.2. Effect of duration of test (Jaworski 1981)

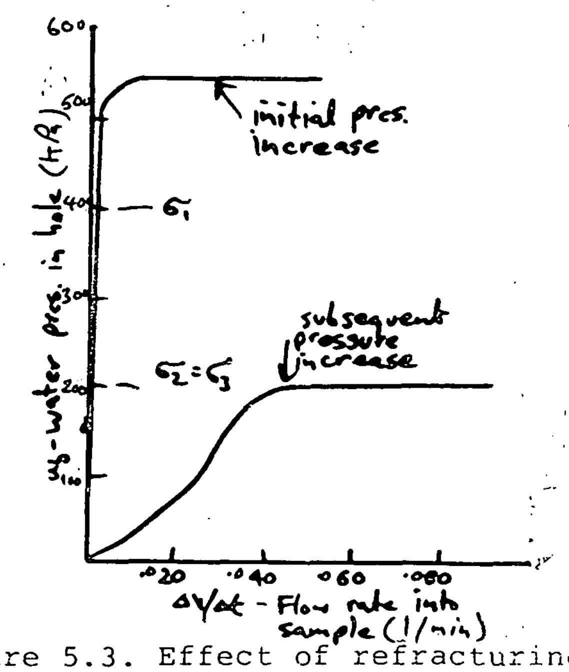


Figure 5.3. Effect of refracturing (Jaworski 1981)

Permeability was noted to increase dramatically at relatively low test pressures, often as low as 0.2 P_o' where P_o' was the effective overburden pressure. Plots of applied pressure vs. apparent permeability were very similar to laboratory results.

Vaughan was involved in similar tests at Balderhead Dam. Hydraulic fracturing of the clay core of this dam occurred during first fill. Later investigations of the results of a remedial grouting programme showed grout filling vertical cracks apparently created by the application of pressure during the grouting operation. Water and mud losses from investigation drill holes also appeared to have been caused by hydraulic fracture. During both grouting and water testing it was noted that fractures, once created, re-opened at pressures less than the original fracture pressures.

Wilkes (1974) carried out hydraulic fracture tests on alluvial deposits under a trial embankment at Kings Lyn in England. Wilkes discussed "the crack (or cracks)" that occurred when excess pressures are applied and attempted to calculate the crack volume. Calculated crack volumes ranged from 46 to 85%, of the total volume of fluid injected during the test. A tendency for the crack to close up after a certain time was explained as due to expansion of the soil on the crack wall.

Wernek et al in measuring K_o of soft plastic soil in Guanabara, Brazil noted a decrease in flow with time once a certain time had been reached. This was interpreted as representing a limitation on the propagation of a crack.

Observations at the Worsley Project during excavations up to 10 metres below natural ground level were that single, vertical fractures occurred at boreholes and extended for several metres from the hole. Crack widths were significant, being up to 3 to 4 millimetres in places.

5.3 MATHEMATICAL MODELS

The understanding of the phenomenon of hydraulic fracture and the development of an adequate, useful mathematical model has progressed slowly from a simple elastic analysis to consideration of an energy balance equation to predict crack propagation (Wong and Farmer 1973).

Even the most rigorous of the current analyses is necessarily simplified and the relationship between actual ground conditions and mathematical assumptions must be carefully considered when making predictions from the models.

Some mathematical models are summarised below. The interest of this thesis and the models reviewed are related to application of hydraulic pressure from a borehole. The borehole is assumed to be vertical and perpendicular to one axis of the principal soil stress distribution.

a) <u>Borehole in Impermeable, Elastic Continum</u>

Fig. 5.4 shows the assumed borehole orientation and defines the geometry.

Initial stress in the ground surrounding the hole is assumed to be the standard solution for stress distribution in a thick cylinder of infinite external radius and zero internal pressure.

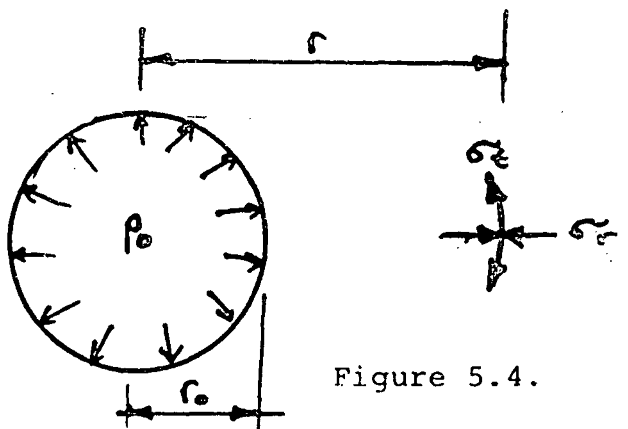


Figure 5.4.

$$\sigma_r = K_o.\gamma.D(1 - \frac{r_o^2}{r^2}) \qquad 5.1$$

$$\sigma_t = K_o.\gamma.D(1 + \frac{r_o^2}{r^2}) \qquad 5.2$$

Where σ_r, σ_t are radial and tangential stresses

K_o is coefficient of horizontal earth pressure

γ is the bulk density of rock or soil

D is the height of overburden

r_o is the radius of the borehole

r is the radial distance from the hole

For internal pressure p_o

$$\sigma_r = K_o.\gamma.D(1 - \frac{r_o^2}{r^2}) + p_o \cdot \frac{r_o^2}{r^2} \qquad 5.3$$

$$\sigma_t = K_o. \ .D(1 + \frac{r_o^2}{r^2}) - p_o \cdot \frac{r_o^2}{r^2} \qquad 5.4$$

Hydrofracture will occur when the tensile stress exceeds the tensile strength S_t of the soil or rock

i.e. $$-[K_o.\gamma.D(1 + \frac{r_o^2}{r^2}) - p_o \cdot \frac{r_o^2}{r^2}] > S_t \qquad 5.5$$

For a material with negligible tensile strength, the conditions at the surface of the borehole are:

$$p_o > 2K_o \ \gamma \ D \qquad 5.6$$

5.6

b) <u>Borehole in a Permeable, Elastic Continuum</u>

For this analysis the fluid force or pressure p_o is considered in two components:

i) that expanding the borehole, and
ii) that inducing seepage flow.

Wong and Farmer defined the proportion of pressures involved in expansion as N with the proportion involved in inducing seepage flow thus being (1-N).

In practice N has been found to lie between 0 and 0.5 for soils and 0.5 to 1.0 for rocks.

From Taylor (1948), fluid pressure at radius r is:

$$p(r) = p_o \frac{\log R/r}{\log R/r_o} \tag{5.7}$$

where R is the radius of the grout front.

The seepage force per unit volume at radius r is given by,

$$-(1-N)\frac{dp(r)}{dr} = \frac{(1-N)\ p_o}{r\ \log R/r_o} \tag{5.8}$$

which has been solved to give,

$$\sigma_R = Np_o\left(\frac{r_o}{r}\right)^2 - (1-N)\ p_o\left[\frac{p(r)}{2(1-\nu)p_o} - F(r)\right] \tag{5.9}$$

$$\sigma_T = -Np_o\left(\frac{r_o}{r}\right)^2 - (1-N)p_o\left[\frac{p(r)}{2(1-\nu)p_o} + F(r)\right] \tag{5.10}$$

where $$F(r) = \frac{1}{2(1-\nu)} \cdot \frac{r_o}{r} + \frac{(1-2\nu)}{4(1-\nu)\log^R/r} \cdot \left(1 - \frac{r_o^2}{r^2}\right) \tag{5.11}$$

σ_R is radial stress from seepage component
σ_T is tangential stress from seepage component

For no vertical strain

$$\sigma_Z = \frac{\nu(1-N)}{(1-\nu)} \cdot p(r) \tag{5.12}$$

At the borehole wall $r = r_o$

$$p(r) = p_o \frac{\log {}^R/r_o}{\log {}^R/r} = p_o \tag{5.13}$$

gives

$$F(r) = \frac{1}{2(1-\nu)} \cdot (\frac{r_o}{r_o}) + \frac{(1-2\nu)}{4(1-\nu)\log(^R/r)} \cdot (1 - \frac{r_o^{\,2}}{r_o^{\,2}})$$

$$= \frac{1}{2(1-\nu)} \qquad 5.14$$

and

$$\sigma_R = Np_o (\frac{r_o}{r_o})^2 - (1-N)\, p_o \left[\frac{p_o}{2(1-\nu)p_o} - \frac{1}{2(1-\nu)}\right]$$

$$= Np_o \qquad 5.15$$

$$\sigma_T = -\, Np_o (\frac{r_o}{r_o})^2 - (1-N)\, p_o \left[\frac{p_o}{2(1-\nu)\, p_o} + \frac{1}{2(1-\nu)}\right]$$

$$= -\, Np_o - \frac{(1-N)p_o}{1-\nu} \qquad 5.16$$

$$\sigma_Z = \frac{\nu(1-N)}{1-\nu} \cdot p_o \qquad 5.17$$

Hydro fracture occurs when the sum of the initial ground forces and the applied grouting forces exceeds the tensile strength of the ground.

i.e., when

$$\sigma_r + \sigma_R \leq -\, S_t$$

$$\sigma_t + \sigma_T \leq -\, S_t$$

$$\sigma_z + \sigma_Z \leq -\, S_t$$

where σ_r, σ_t and σ_z are initial ground stresses.

As σ_r, σ_R are always compressive, only horizontal or vertical fractures are possible.

Vertical fractures are initiated when:

$$\sigma_t + \sigma_T = -\, S_t$$

i.e, $$2\,K_o\,\gamma\,D - Np_o - \frac{(1-N)}{(1-\nu)}\cdot p_o = -\,S_t$$

or $$\frac{p_o}{\gamma D} = \frac{1-\nu}{1-N\nu}\,(2\,K_o + \frac{S_t}{\gamma h}) \qquad 5.18$$

Horizontal fractures are initiated when:

$$\sigma_z + \sigma_Z = -\,S_t$$

i.e, $$\gamma D - \nu\frac{(1-N)}{(1-\nu)}\cdot p_o = -\,S_t$$

or $$\frac{p_o}{\gamma D} = \frac{1-\nu}{\nu(1-N)}\cdot(1 + \frac{S_t}{\gamma h}) \qquad 5.19$$

From these latter two equations, 18 and 19, the various conditions for hydrofracture can be deduced.

These are summarised in Figures 5.5 and 5.6 assuming tensile strength is negligible.

For soils, common values of the various parameters are:

ν = 0.2 to 0.5
K_o = 0.5 to 1.5
N = 0 to 0.5

From substitution of these figures into Figures 5.5 and 5.6 it can be seen that hydrofracture is often possible at grout pressures well below overburden pressures and that vertical cracking is favoured under most common conditions.

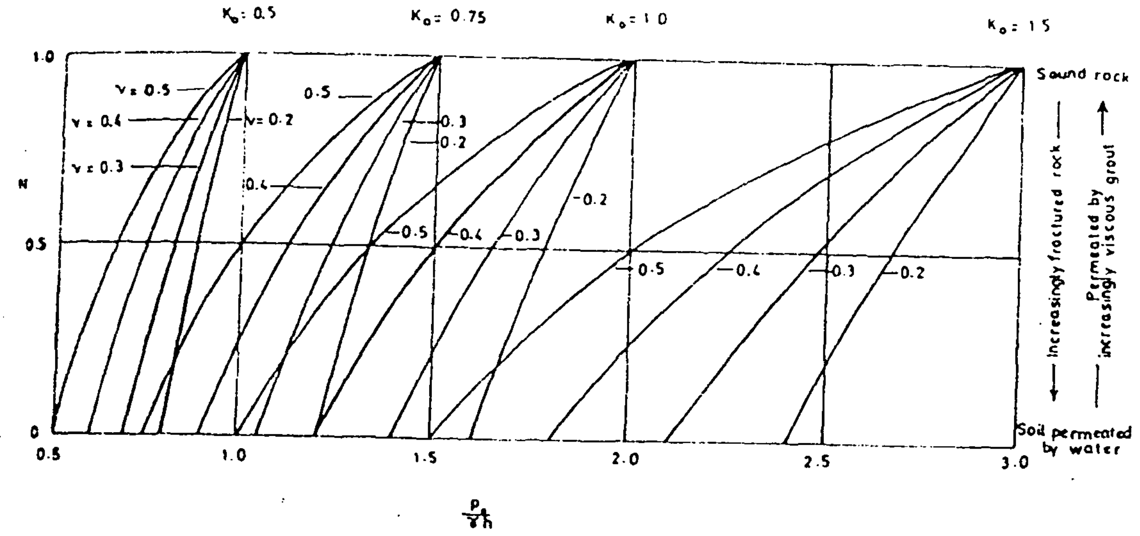


Figure 5.5. Conditions for initiation of vertical cracks (Wong & Farmer)

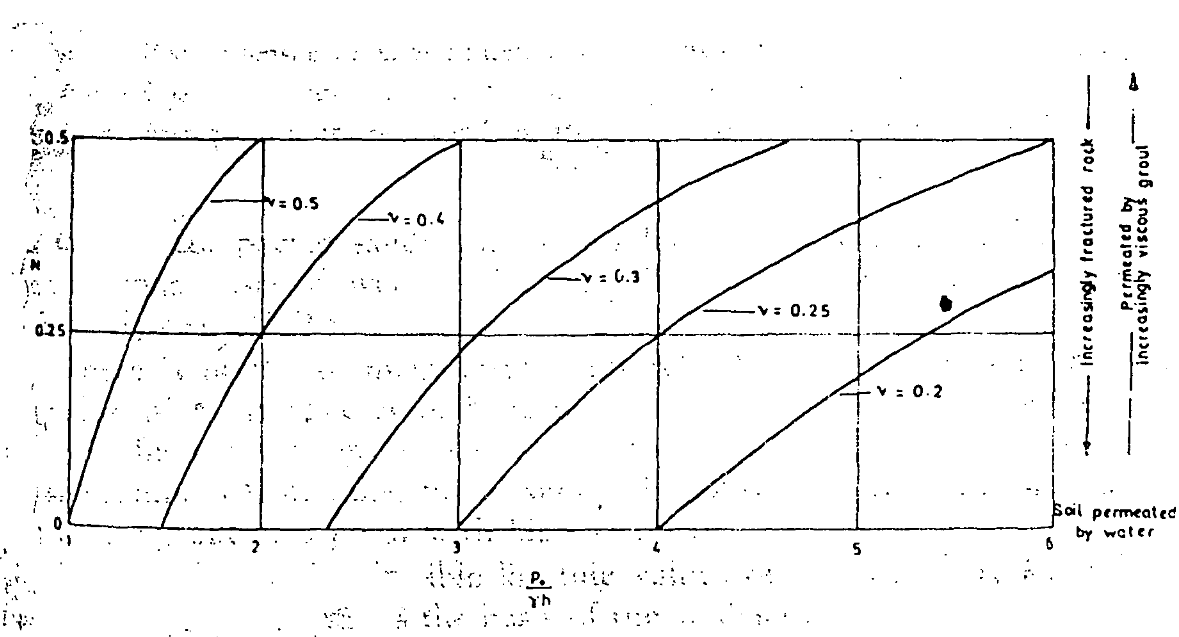


Figure 5.6. Conditions for initiation of horizontal cracks (Wong & Farmer)

c) <u>Borehole Surrounded by a Post-fracture Non-elastic Zone</u>

Following initial fracture the elastic model is reconstructed as developing three distinct zones, namely:

i) A non-elastic, fractured zone around the hole.

ii) An elastic zone with grout flowing.

iii) An ungrouted zone.

The geometry of this case is shown in Fig. 5.7

Wong and Farmer (1973) suggest that this situation can be analysed as a thick cylinder subject to an internal force of Np_o, a body force of $\dfrac{(1-N)p_o}{r\log^R/r_o}$, and a horizontal containing force due to overburden pressures.

In the inelastic zone, considering the equilibrium of an elementary volume $dr.rd\theta$ gives

$$r \cdot \frac{d\sigma_r}{dr} + (\sigma_r - \sigma_t) = \frac{(1-N)p_o}{\log(^R/r_o)} \qquad 5.20$$

where $\sigma_t = \beta\sigma_r - \bar{c}$

and $\beta = \tan^2(^\pi/4 - ^\phi/2)$

$\bar{c} = \dfrac{2\cos\phi}{1+\sin\phi} \cdot S_t$

ϕ = angle of internal friction

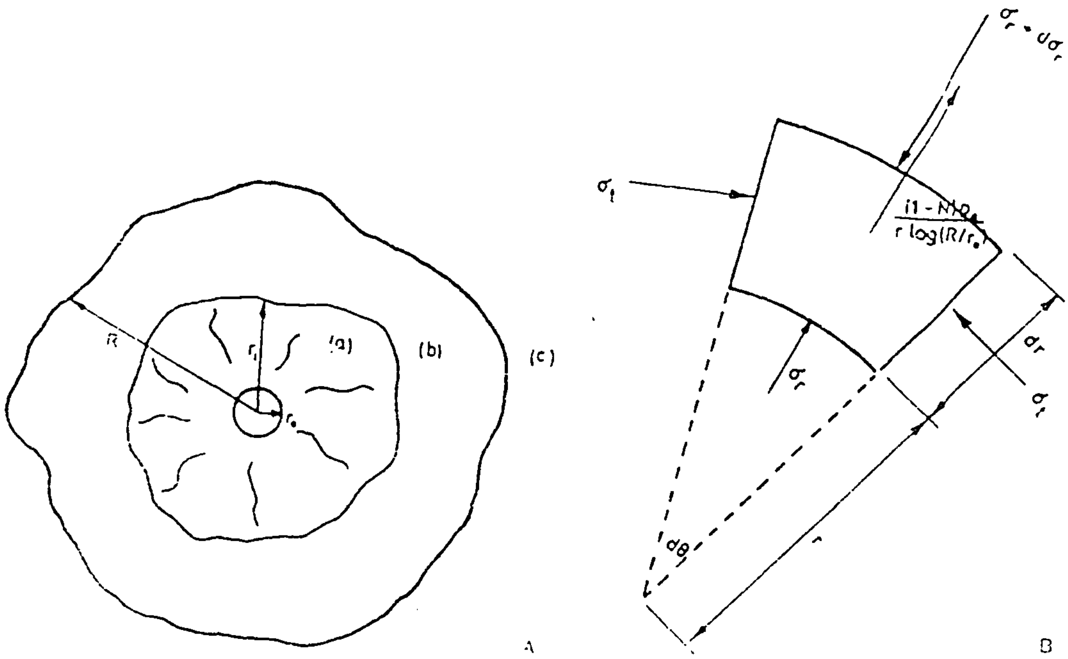


Figure 5.7.
A   Zones around aborehole
a)Fractured zone
b)Elastic-grout flow
c)Elastic-no grout flow

B   Stresses acting on a soil element

Solving for pressure p_o required to produce a plastic zone of radius r_1

$$p_o = \frac{2K_o \gamma D + \bar{c}\ \{(1-\beta)[1-(\frac{r_1}{r_o})^{\beta-1}]/(1-\beta)+1\}}{(1+\beta)N(^{r_1}/r_o)^{\beta-1} + [\frac{(1+\beta)(1-N)}{(1-\beta)\log(^R/r_o)}][1-(\frac{r_1}{r_o})^{\beta-1}] + \frac{(1-N)\log^R/r_1}{(1-\nu)\log^R/r_o}}$$

5.21

for $S_t = 0$

get $\bar{c} = 0$

and

$$p_o = \frac{2\ K_o\ \gamma\ D}{(1+\beta)N(^{r_1}/r_o)^{\beta-1} + [\frac{(1+\beta)(1-N)}{(1-\beta)\log(^R/r_o)}][1-(\frac{r_1}{r_o})^{\beta-1}] + \frac{(1-N)\log^R/r_1}{(1-\nu)\log^R/r_o}}$$

5.22

Figure 5.8 shows the effect of this equation on some typical examples of soils and rock.

The solution is very much dependent on the relationship between the radius of the grout hole r_o, the radius of the fracture zone r_1 and the radius of the grout envelope R.

For less permeable soils where the rate of advance of the grout front is slow then R can be considered equal to r_1. Further, in the limiting case, when the zone of fracture can be considered as an effective increase in the grout hole diameter.

$$p_o = \frac{2\ K_o\ \gamma\ D}{(1-\beta)N + \frac{1-N}{1-\nu}}$$ 5.23

which is constant.

The only restraint in this case, on the nature and extent of crack propagation is the energy available from the grouting equipment.

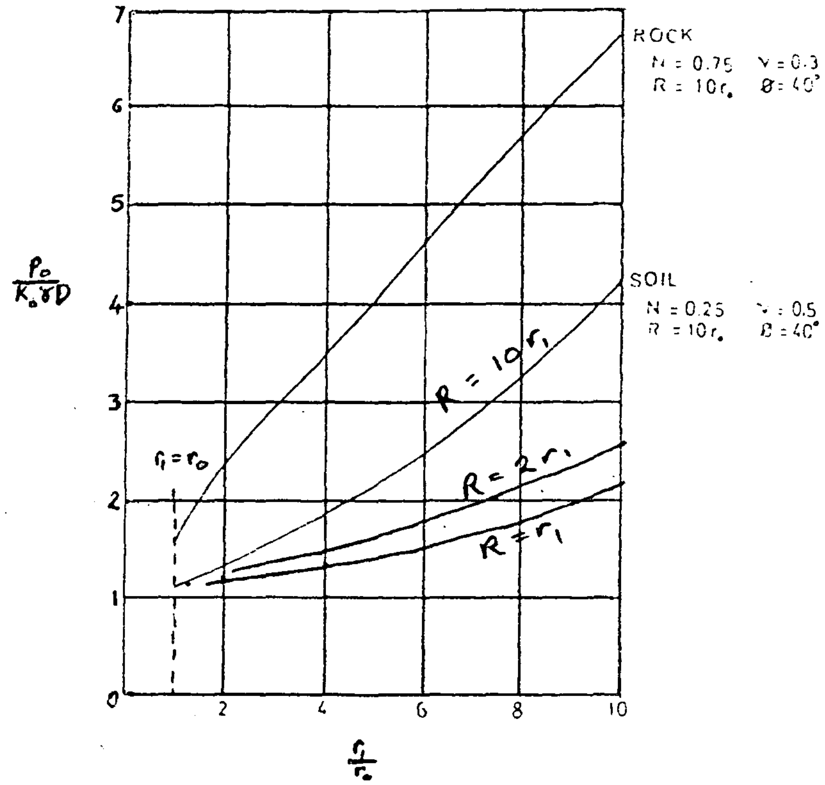


Figure 5.8. Propagation of Hydrofracture
(Wong & Farmer)

d) Alternative Method, Expansion of a Cylindrical Cavity

Massarch (1978) presents an alternative model based on the theory of an expanding cylinder developed by Bishop et al (1949).

Figure 5.9 defines the geometry.

Massarch claims that for undrained conditions:

$$\frac{p_o}{\tau_f} = \ln\left[\frac{1.36E}{\tau_f(1+\nu)}\right] \qquad 5.24$$

$_o$ = fluid pressure to expand cavity
τ_f = undrained shear strength
E = Youngs Modulus
ν = Poissons ratio
r_1 = radius of plastic zone
r_o = radius of borehole

The extension of the plastic zone is given by:

$$\frac{r_1}{r_o} = \left[\frac{E}{2(1+\nu)\quad\tau_f}\right]^{\frac{1}{2}} \qquad 5.25$$

Total stresses at radius r are given by:

$$\frac{\Delta\sigma_r}{\tau_f} = 2\ln {}^{r_1}/_{r} + 1 \qquad 5.26$$

$$\frac{\Delta\sigma_t}{\tau_f} = 2\ln {}^{r_1}/_{r} - 1 \qquad 5.27$$

$$\frac{\Delta\sigma_z}{\tau_f} = 2\ln {}^{r_1}/_{r} \qquad 5.28$$

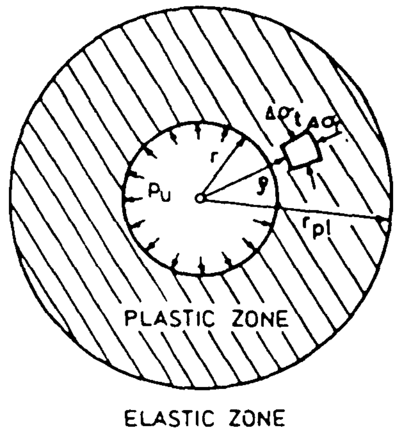


Figure 5.9. Stress Conditions in Plastic Zone Around Expanding Cavity

and excess pore pressure is given by:

$$\frac{\Delta u}{\tau_f} = 1.733\ A_f - 0.577 + 2 \ln {}^{r_1}/_r \qquad 5.29$$

Where A_f = Skemptons pore pressure parameter

Effective stresses become:

$$\frac{\Delta\sigma_r'}{\tau_f} = 1.577 - 1.733\ A_f \qquad 5.30$$

$$\frac{\Delta\sigma_t'}{\tau_f} = -0.428 - 1.733\ A_f \qquad 5.31$$

$$\frac{\Delta\sigma_z'}{\tau_f} = 0.577 - 1.733\ A_f \qquad 5.32$$

For soils with negligible tensile strength fracturing will occur when:

$$\sigma_r' + \sigma_t' = 0 \qquad 5.33$$

i.e.

$$\frac{\sigma_v'\ K_o}{\tau_f} < 1.73\ A_f + 0.43 \qquad 5.34$$

from equation 5.23:

$$\frac{p_o}{\sigma_v' K_o} < \frac{\ln\left[\frac{1.36E}{\tau_f(1+\nu)}\right]}{1.73\ A_f + 0.43} \qquad 5.35$$

e) <u>Energy Balance Concept</u>

An interesting approach to the problem is the "Energy Balance" concept proposed by Perkins and Krech, (1968) which postulates that the energy supplied by the grout injection pump must equate with the various areas of work done by the grout in the soil. These areas are summarised below:

1. Recoverable energy (Er)

i) Elastic strain energy stored in rock or soil.

ii) Elastic strain energy stored in the fluid.

2. Irrecoverable energy (Ei)

i) Work done in fracturing rock or soil.

ii) Work done to cause plastic deformation of the fractured zone.

iii) Work done to overcome frictional drag between fluid and rock or soil surfaces during flow.

iv) Work done to overcome the shear strength of the fluid during flow.

v) Work done to overcome various frictional forces in the grouting system.

For any time interval the additional energy supplied will be given by:

$$E = p_{o_{av.}} \Delta V$$

where $p_{o_{av.}}$ is the average grout pressure and ΔV is the volume of grout injected over the time interval.

As the fracture zone or grout front expands an increasing flow rate of grout from the pump will be required to maintain conditions. Hence the propagation of fractures will be limited by the physical capacity of pumps being used.

5.4 APPLICATION USING WORSLEY SOIL PARAMETERS

To demonstrate the effect of the various theoretical models, parameters from the Worsley soils will be used to calculate the hydraulic fracture pressures predicted by each model.

Relevant parameters which have been given in Section 2 are summarised below.

K_o = 0.36 (minimum recorded)
γ = 19 kN/m^3 typical
ν = 0.4
Assume St = 0
ϕ = 30^o
E = 26 MPa
τ_f = 0.42 MPa

Other parameters derived by assumption are:

N = 0.5
A_f = 0.8
σ_v = γh

Inserting these values into the various formulae gives:

a) Borehole in Impermeable Elastic Continuum

Substitution into equation 5.6 gives:
p_c = 13.7 h

b) Borehole in Permeable Elastic Continuum

Substitution into equation 5.18 for vertical fractures gives:

p_o = 10.3h

Substitution into equation 5.19 for horizontal fractures gives:

p_o = 57h

c) <u>Borehole with Plastic Zone</u>

Substitution into equation 5.23 gives:

p_o = 9.13h

d) <u>Expansion of a Cylindrical Cavity</u>

Substitution into equation 35 gives:

p_o = 18.6 h

Predictions are thus that vertical hydraulic fractures could occur at grout pressures of between 9.1h and 18.6h.

With actual overburden pressure being estimated at 19h this demonstrates that vertical hydraulic fracture could occur at grout pressures as low as half overburden pressure.

CHAPTER 6 - GROUTING OF SOIL FOUNDATIONS IN DAM CONSTRUCTION

6.1 DISCUSSION

A major use of grouting technology is in the improvement of dam foundations. The primary aim of grouting in dam construction is the reduction of seepage through the foundations for the purpose of either:

(i) preventing loss of valuable water

(ii) reducing the potential for 'piping' or wash out of foundation materials

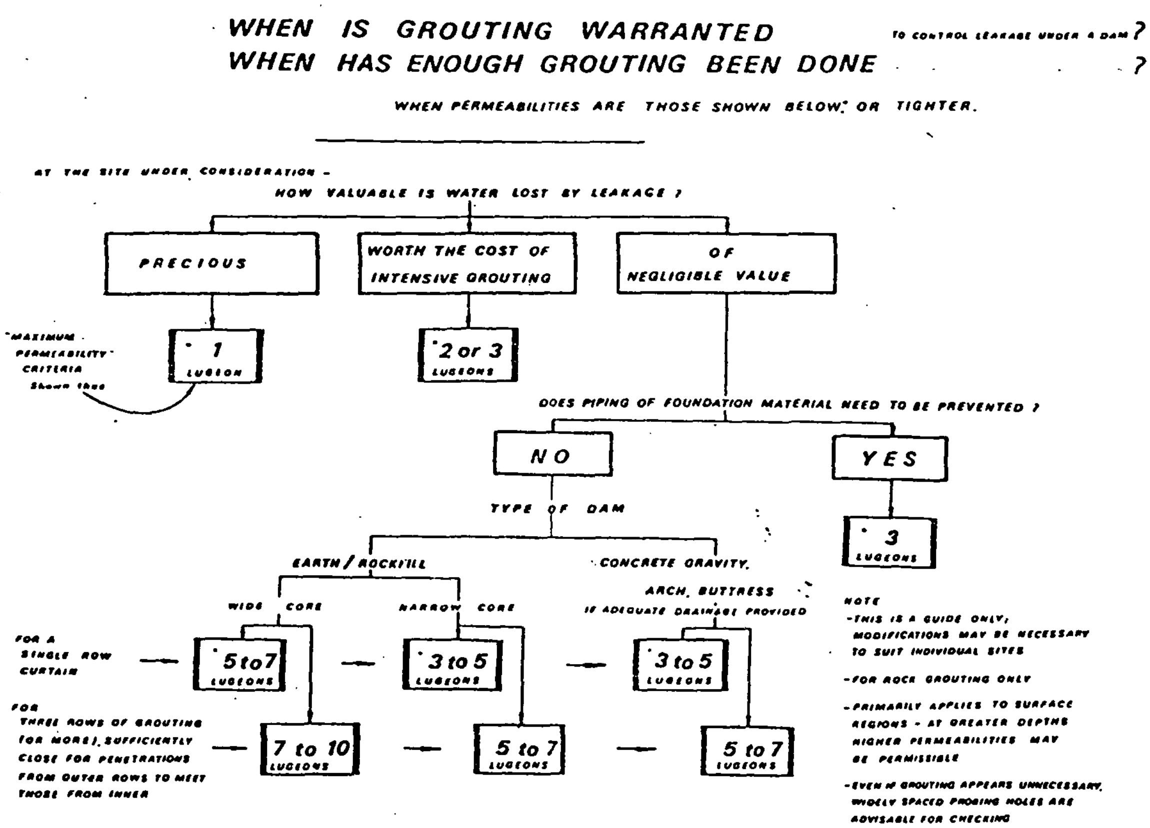
(iii) reduction of uplift pressures on the downstream side of a dam to increase stability.

Fig 6.1 from Houlsby (1977) suggests guidelines to the need for foundation grouting in various conditions. The lugeon value used by Houlsby is defined as the flow measured in a foundation permeability test in litres per minute per metre of hole at a test pressure of 10 bar (1035 KPa). This unit is usually only used in relation to the permeability of rock but can be related to soil permeability by the approximate relationship:

1 lugeon = 1.3×10^{-7} m/sec

As can be seen, Houlsby recommends that grouting should normally be carried out when foundations are more permeable than 10^{-7} metres per second and should always be carried out when permeabilities are greater than 10^{-6} metres per second.

There is, however, a certain amount of controversy over the effectiveness of grout curtains, particularly if reliance is placed on a single, relatively thin curtain.



Flow Chart Indicating Suggested Standards for Grouting

Figure 6.1. (after Houlsby)

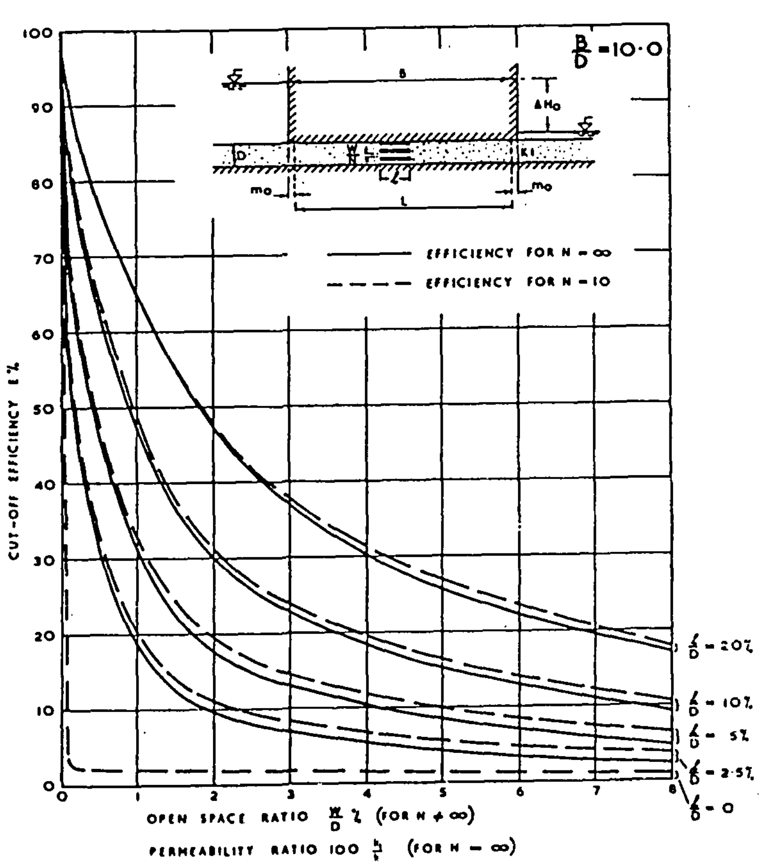


Figure 6.2. Grout Curtain Efficiency
(after Ambraseys)

Dachler (1926) produced an analysis expressing the efficiency of thin grout curtains, as

$$E_c = \frac{\ln \sin \frac{\pi W}{2D}}{\ln \sin \frac{\pi W}{2D} - \frac{n \pi B}{4D}} \qquad 6.1$$

where E_c in the percentage reduction in seepage flow caused by the curtain

W = total width of space in curtain

D = depth of curtain

B = width of dam base

Inserting some hypothetical values into the equation indicates the degree of sealing necessary for a high efficiency cut off. An example used by Professor Arthur Casagrande in the first Rankine Lecture considered a curtain equivalent to an impermeable membrane with 1/16 inch (1.5mm) slots at 5 ft (1.5m) intervals over a total depth of 100 ft (30m). This results in an open space ratio of 0.1%. Assuming B = D = 100 ft, Z = 20, W/2 = 1/16 inch, Cassagrande calculated E_c = 29%.

Ambraseys (1963) further developed this concept to allow consideration of thick curtains, resulting in the relationship.

$$\frac{100[L(^k/k_1-1)/D]}{B/D + L(^k/k_1-1)/D + 0.88} \% \qquad 6.2$$

where L = thickness of grout curtains

k = permeability of the ground

k_1 = permeability of the grout curtain

Figure 6.2 shows the effect of various parameters on the cut-off efficiency. It is apparent that for smaller B/D ratios, the greater is the effect of the cut-off thickness.

Grout curtains are thus normally constructed as multiple row curtains, often with the number of rows increased over the upper part of the curtain to decrease the hydraulic gradient across the curtain in the most critical areas.

Some typical grout curtain arrangements are shown in Fig. 6.3 (from Houlsby, 1982).

The majority of dams constructed in recent times have included a grout curtain in their design. The majority of dams however have been constructed on rock foundations and only cement or clay-cement grouting has been carried out.

Documented cases of grout curtains in soil foundations are very limited particularly using pure solution grout, and despite considerable research, only a few examples are available for review.

Three examples described relate to curtains constructed in relatively permeable foundations using cement/clay-cement/deflocculated clay/silica gel grout systems. Whilst not involving pure solution grout the projects described indicate the essential features of grouting in soils.

A further example describes the use of the pure solution grout AM-9 in post construction grouting of permeable layers in an embankment.

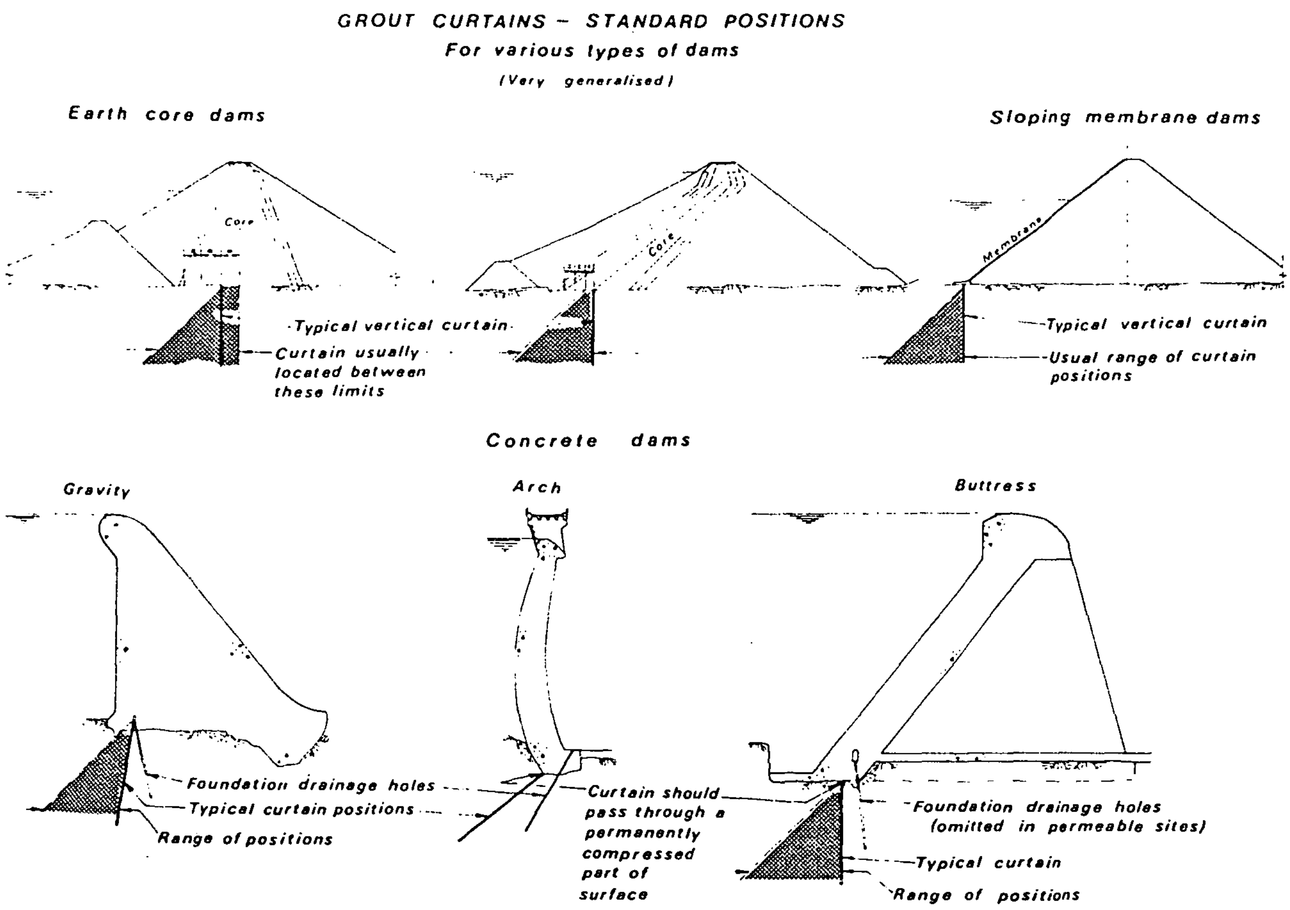


Figure 6.3. Typical Grout Curtain Positions
(Houlsby 1982)

6.2 Examples of Grouting of Soil Foundation in Dam Constructions

6.2.1 Durlassboden Dam

The grouting at Durlassboden Dam carried out in Austria in 1967 is described by Caron (Winterkorn and Fang). The 70m high earth fill dam was founded on alluvial deposits in the valley floor.

These comprised up to 50 metres of sands and gravels with permeability of around 10^{-4}m/s overlying silts with a permeability of 10^{-5} to 10^{-6}m/s and fine sands of 10^{-5} metres per second down to a total depth of nearly 150m.

Seepage considerations determined that the foundations would be suitable provided that seepage through the upper sands and gravels could be controlled by a method which would be able to accommodate the expected considerable post construction settlements.

A grout curtain was designed as shown in Fig. 6.4 with three phases of grout injection through tubes a manchette. These phases were:

Phase 1 injection 22,540m^3 of clay/cement grout

Phase 2 injection 22,132m^3 of bentonite gel (comprising bentonite, sodium monophosphate, sodium silicate and water with viscosity 38 Marsh seconds, or 14 centipoise).

Phase 3 injection 6,724m^3 of "Algonite" gel (comprising mainly silicate aluminate with a viscosity of 32 Marsh seconds, or 6 centipoise).

Rows were 3 metres apart with holes at 3 metre centres final grout pressures were up to 5,000 kPa at depth.

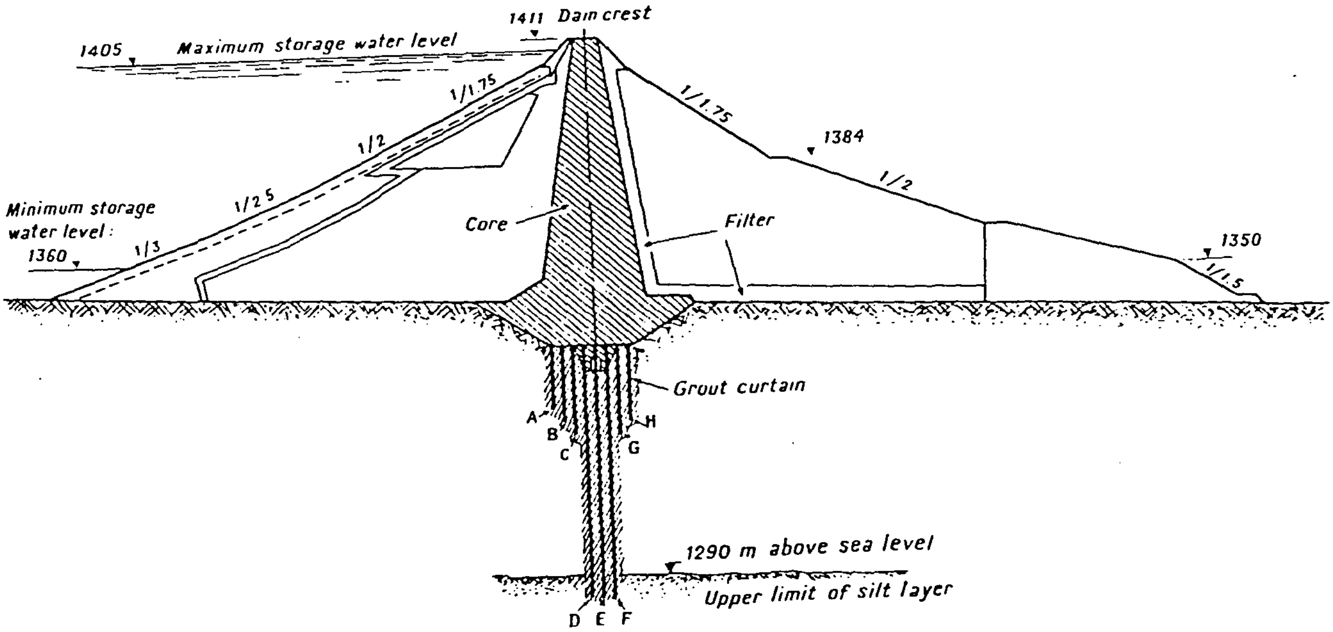


Figure 6.4. Durlassboden Dam Grout Curtain

The work was carried out by French grouting contractors "Soletanche".

Calculations assuming a radius of grout travel of 1.5 metres indicated a total grout absorption of 45% of the soil volume.

6.2.2 Backwater Dam

This dam constructed in Scotland in 1966 comprised an earth embankment 43m high and 549 metres long over a permeable foundation of sand, gravel and glacial tills to 50 metres deep with a permeability of 10^{-4} to 10^{-6}m/s.

A grout curtain comprising up to 5 rows of holes was designed utilising 40mm diameter steel tubes a manchette.

Three phases of injection were carried out, namely:

Phase I - clay/cement grout to outer rows (viscosity 12 to 14 centipoise)

Phase II - deflocculated clay (viscosity 12 to 14 centipoise)

Phase III - silicate based grout (viscosity 6 to 7 centipoise)

Holes and rows were 3m apart and initial trial injection volumes approximated 25% of soil volume injected. However, vertical heave of 8 inches was measured over a test section implying that excessive grout pressures were being used. It was realised that, particularly in the till material with a natural permeability of 10^{-7} m/second, this over injection by rupture could be dangerous and create leakage paths. Injection volume was subsequently reduced to 3.75% of soil volume in the till areas.

Permeability reduction measured after the completion of phases 1 and 2 was 15 times and after phase 3 was 300 times.

Grouting pressures were up to 1000 KPa for the 50 metre deep curtain.

The success of the operation was judged by the development of a 6m head loss across the curtain during a pump test.

6.2.3 Notre Dame de Commiers

This earthfill dam constructed on the River Drac in south east France, is described by Bonazzi (1965).

The dam is 40 metres high with a crest length of 300 metres. The foundation in the river bed comprised 50 metres of highly pervious alluvium deposit with an approximate area of 7,000 sq.m.

A typical X-section is shown in Fig. 6.5. Permeabilities were determined by numerous injection tests and ten pump out tests are different levels, giving results as follows:

Depth (m)	Permeability (m/sec)
0 to 4	10^{-1} to 10^{-2}
4 to 10	2×10^{-2} to 4×10^{-3}
10 to 26	2×10^{-3}
below 26	3×10^{-4}

A grouting programme was carried out by French contractors "Etabissements Soletanche" and consulting engineers "Coyne et Bellier" with a contract specifying the degree of watertightness to be achieved.

The curtain comprised five rows of holes at 3 metre centres, 3 metres apart.

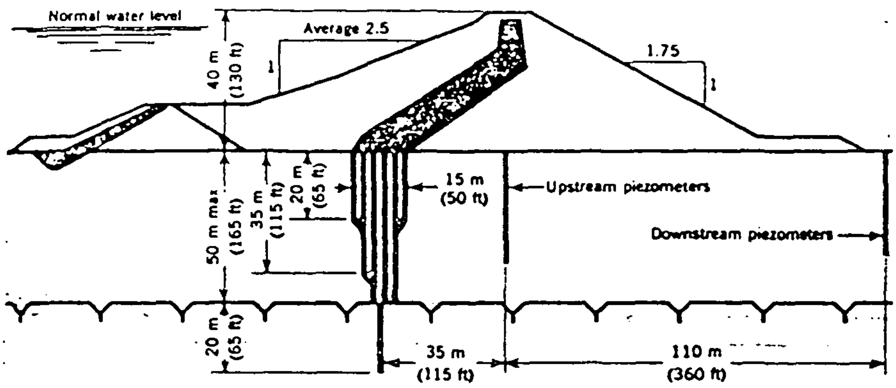


Figure 6.5. Notre-Dame de Commiers Dam

Three types of grout were used, namely clay cement, deflocculated clay and silica gel injected individually in the order stated.

A fixed volume of grout was injected, calculated to account for 30% to 40% of the expected volume of soil to be grouted. Grouting proceeded from the outer rows to the inner rows. The tube a manchette technique was used.

A total of 13,000 tonnes of clay cement, $6000m^3$ of deflocculated clay and $2000m^3$ of silica gel were used for the treatment of a theoretical grout curtain volume of $90,000m^3$.

The success of the project was measured by review of the water table conditions over the entire site both before and after grouting. These showed a far greater degree of sealing than had been specified.

6.2.4 Parangana Dam

The Parangana Dam is an earth and rockfill dam constructed by the Hydro Electric Commission of Tasmania as part of the Mersey Forth Power Development.

During construction in 1967 two areas of concern related to earth core and the downstream filter zone led to the use of the acrylamide grout AM-9. The work is described in an internal report made available to the author.

When construction of the embankment had reached a height of approximately 35 feet (10 metres) grading tests on the materials used in the downstream 2B filter zone indicated that it was unsuitable as a filter to the zone 1A core material. It was decided to seal the material with AM-9.

At the same time permeable features were detected in the zone 1A material. These were assessed to be horizontal fissures formed by the action of compaction equipment. A grout curtain was proposed to seal these features.(Fig.6.6.)

Grouting of the 2B zone was carried out by the lost point method in which a grout pipe is driven to the required depth and then withdrawn slightly to expose a section of open hole for grouting. Three rows of holes were driven, each 300 millimetres apart. Holes were at 525 millimetre centres.

Injection took place in a primary/secondary sequence.

Stage lengths were 450 millimetres.

All stages were water tested and chemical grouting only carried out if a take was recorded. Stages in the outer rows were grouted with 40 litres of grout per stage and those in inner rows were grouted to refusal, defined as 3 litres per metre per minute.

Initial grouting was carried out at a pressure of 20 p.s.i. (640 KPa) but excessive surface leakage, probably due to hydraulic fracture caused the pressure to be reduced, particularly at shallow depths..

The results of 2B grouting showed a clear reduction in take over the inner row and indicated apparent success of the grouting operation.

Grouting of the 1A zone took place from augered holes at 900 millimetre centres. Alternate holes were grouted as primaries and secondaries with an inflatable packer being placed near the top of the hole. The injection pressure was reduced from 20 p.s.i. (640 KPa) to 10 p.s.i. (320 KPa) due to excessive surface leakage. Holes were grouted for 15 minutes.

Results were not conclusive. However secondary hole takes were in general less than primary hole takes and it was concluded that the operation had been successful.

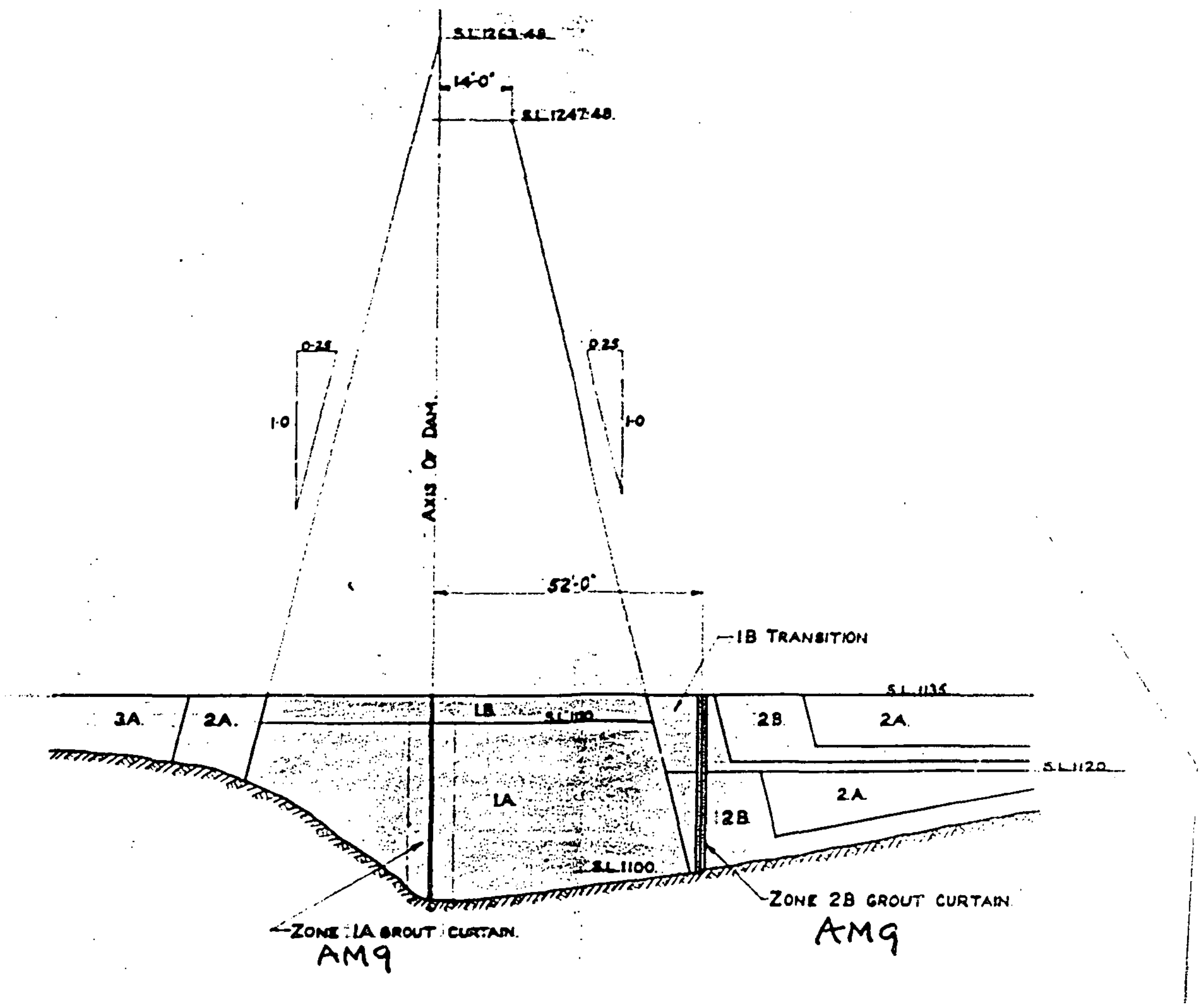


Figure 6.6. X-Section, Parangana Dam core and filter grouting

CHAPTER 7 - GROUTING OF THE REFINERY CATCHMENT LAKE DAM

7.1 DESIGN PHASE GROUTING TRIAL

7.1.1 Introduction

Following the indication, from a seepage analysis, that a grout curtain beneath the Refinery Catchment Lake Dam at the Worsley Alumina Refinery could effectively prevent seepage of contaminated water past the downstream collection systems, various field grouting trials were carried out in conjunction with detailed literature search and design work.

A programme of test grouting was designed to investigate the performance of cement and chemical grouts in the various weathered rock profiles of the site. It was anticipated that cement grout would be effective in only the most permeable sandy or highly fissured soils whereas the chemical grouts would penetrate and seal the relatively low permeability sandy silts and silty clays common to the area.

7.1.2 Grout Types

The grouts chosen for test were

(a) Cement: Type B Portland cement at an initial mix ratio of 10 parts of water to 1 part cement was used. Cement grouting is commonly used for a wide range of grouting applications and its properties are well known. It is extensively used in the sealing of joints in rock.

The possible use of cement grout in the overburden soils was expected to be to seal any large fissures or highly permeable zones. If further reduction of the permeability was then required, less volume of the more expensive grouts would be used.

(b) Sodium Silicate: Sodium silicate comes in the form of a syrupy liquid with its viscosity varying with the water content and the ratio of SiO_2 to Na_2O. The grout is actually a basic colloidal solution and gelling takes place on neutralising with an acid.

The type used was Vitroseal Grade N50 with MEGDA catalyst manufactured by ICI, with a viscosity of 300 centipoise at 20°C. Grout and catalyst were delivered to site in 200 litre drums and mixed with conventional grouting equipment. The mix proportions were found to be critical to the successful setting of the grout and gel time was greatly influenced by temperature.

Weather conditions on site during trial grouting resulted in the grout reaching temperatures of up to 30°C during mixing and injection. Gel time ranged between 15 and 25 minutes, however the supplier advised that up to 90 minutes may be achieved in cooler conditions. The short gel time rendered grouting impractical as injection needed to be continually interrupted and equipment cleaned.

(c) Pure Solution Grout: A low viscosity, low toxicity grout was considered ideal for the application and a phenoplast resin Geoseal MQ4, manufactured by Borden Chemicals was selected. Acrylate AC400 would also have been a suitable grout for trials but was unavailable at the time of the tests.

The geoseal resin and catalyst (caustic soda) are supplied in powder form in pre-weighed bags. These are easily mixed with water using conventional grouting equipment. Viscosity at 25°C was quoted at 2 centipoise for a 12.5% solution.

Temperature was found to be extremely critical during the mixing stage and ice was required to keep the temperatures within the specified limits. Gel time was kept in the 4 to 5 hour range allowing time to grout several stages without the need for clean up of equipment.

7.1.3 Test Site and Results

Three sites were designated for testing.

At site G1 grouting was carried out with both cement and silicate in material with an average permeability of 2 x 10^{-6}m/sec. Site G2 was grouted with silicate and resin grouts under similar conditions, whilst at Site G3 cement grouts were tested in an area with suspected high permeability zones.

Each test site comprised a central test hole, with slotted PVC casing isolated over the strata to be tested, surrounded by four equally spaced primary holes.

Pressure testing of the central test hole was carried out before and after grouting operations in an attempt to quantify results. Where thought useful, secondary and tertiary holes were placed between primaries to enable further evaluation of grouting success.

The sequence of events at each site and the results obtained are represented diagramatically on Figs 7.1, 7.2 and 7.3.

It should be noted that the permeabilities measured are the net result over several metres of soil in which a small proportion of highly permeable seams may exist in a relatively impermeable mass.

All holes were drilled using rotary wash boring techniques and were cleaned by water flushing. Grouting was carried out at 100 kPa pressure to refusal.

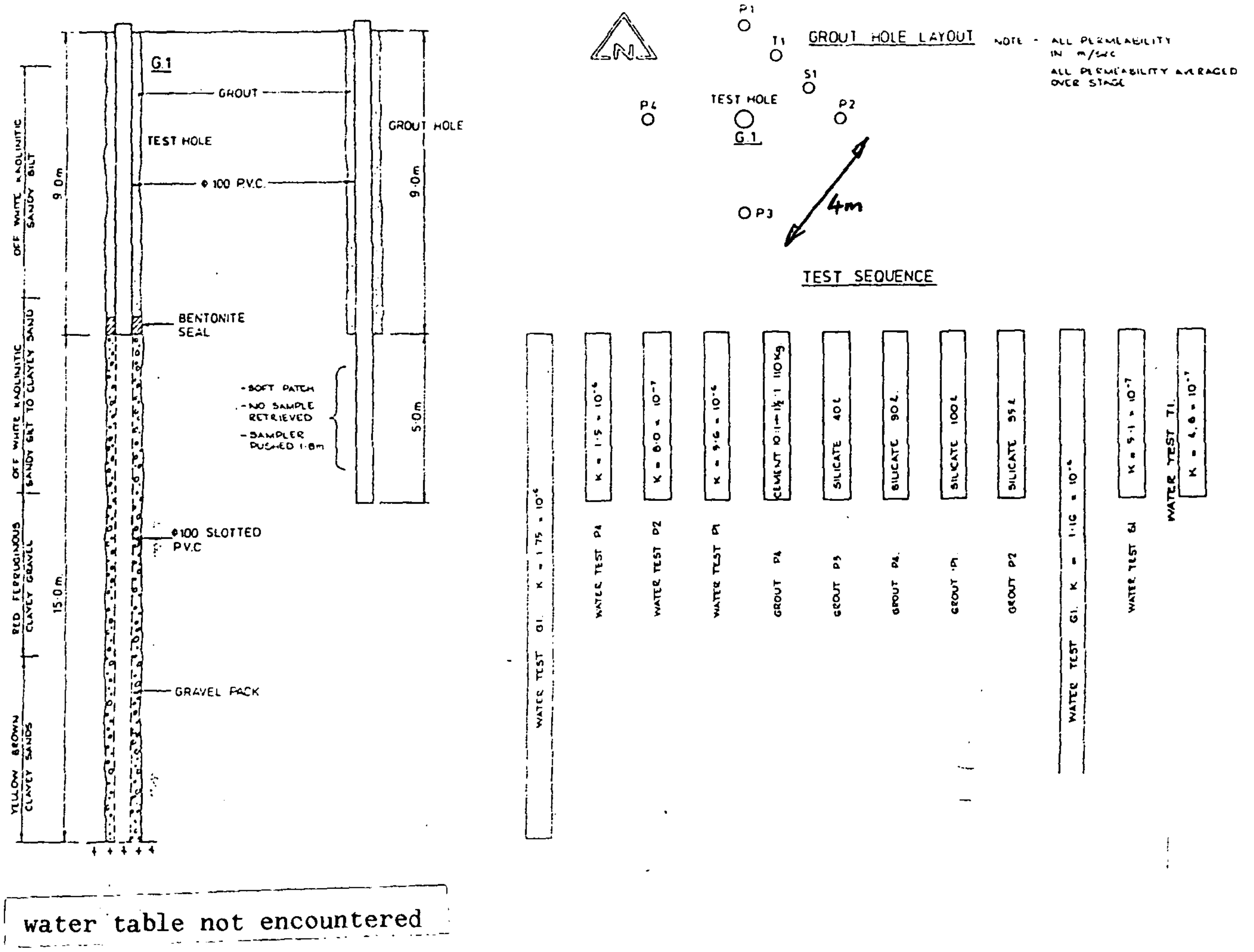


Figure 7.1. Test Grouting Site G 1

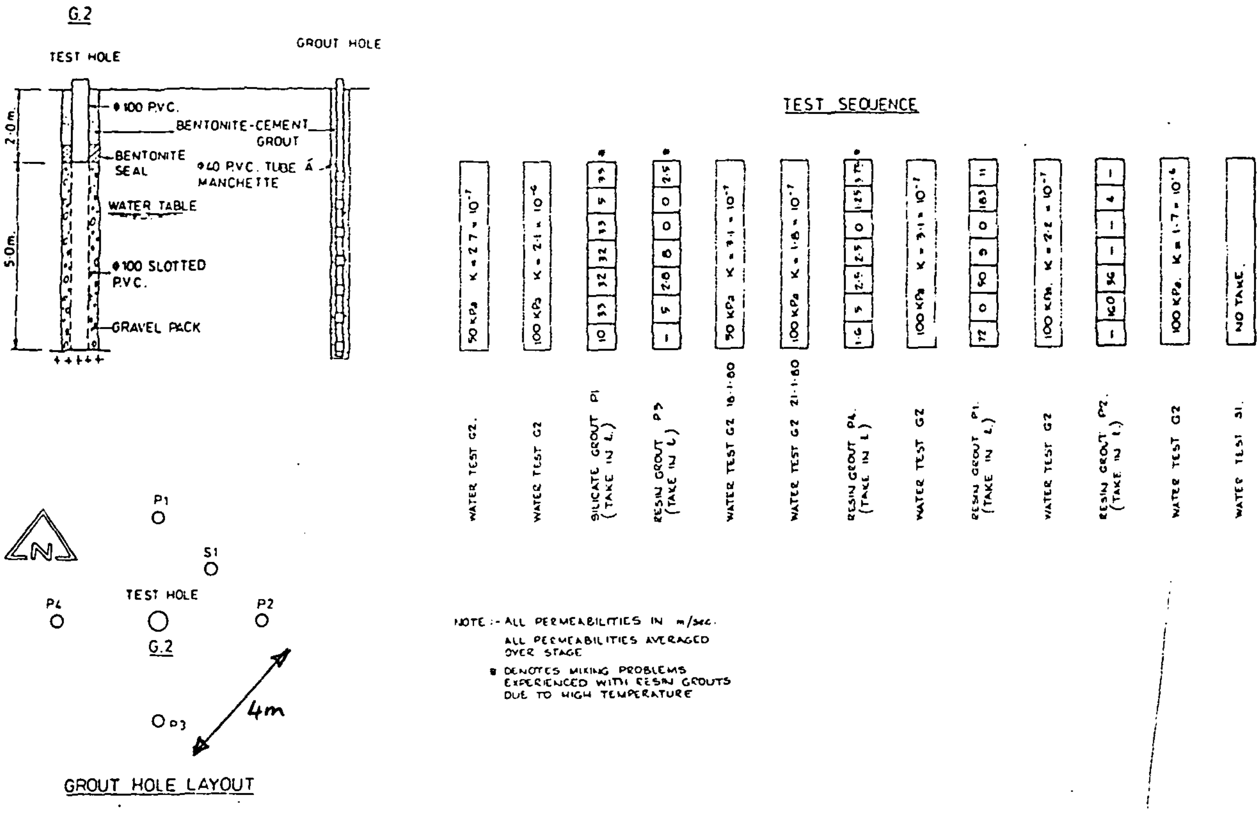


Figure 7.2. Test Grouting Site G 2

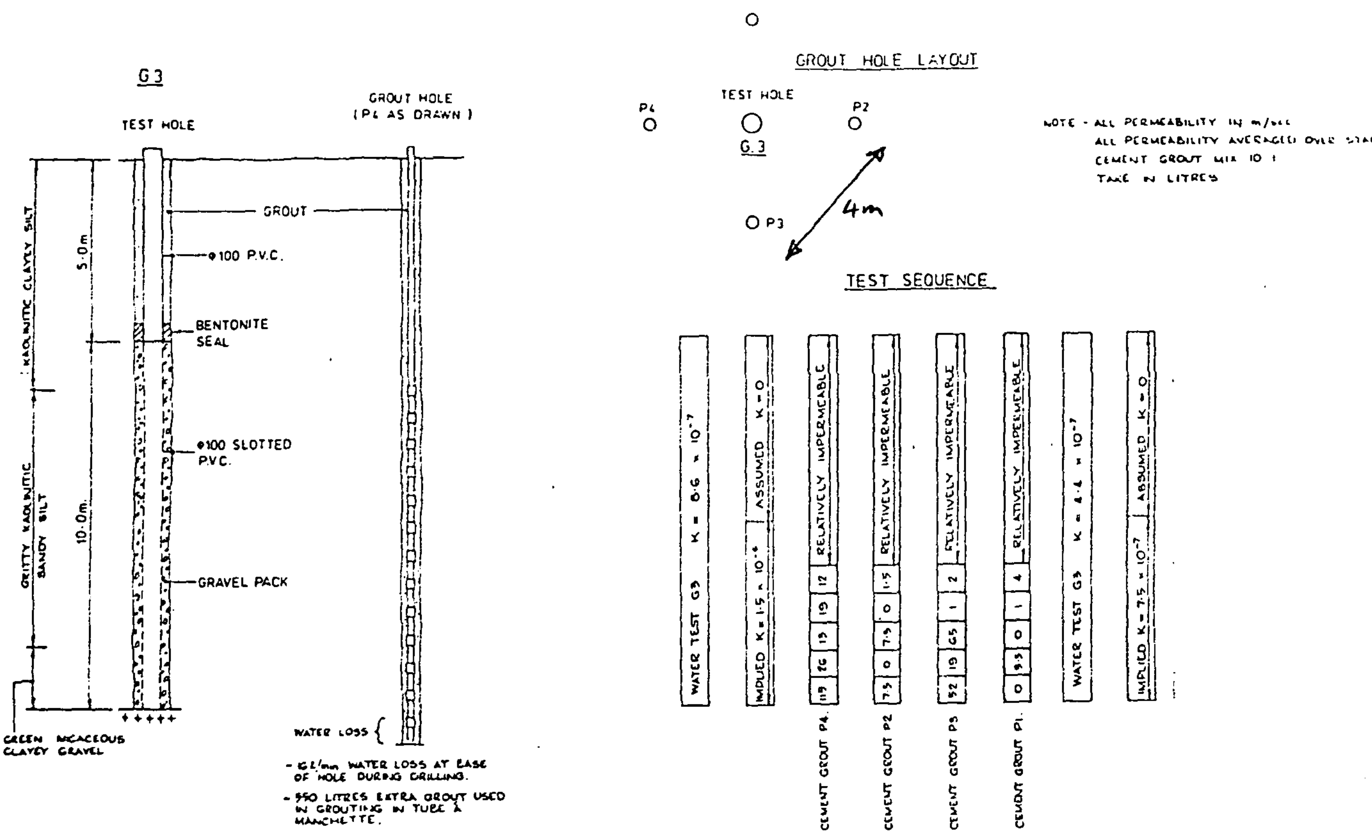


Figure 7.3. Test Grouting Site G 3

7.1.4 Summary of Results

Financial and practical considerations prevented the achievements of conclusive quantitative results from the trial programme. Nevertheless the programme did demonstrate the properties of the grouts used.

At site G1 grout holes could not be extended to the expected depth due to caving in of soft material at a depth of 14 metres. The central test hole could thus not be effectively used to review permeability changes. However, the permeability of soil in the depth range 9 metres to 14 metres was measured as averaging 2×10^{-6} and injection of a thin cement grout and a silicate grout into each primary hole proved possible. Testing of intermediate holes S1 and T1, suggested that the cement/silicate combination had reduced permeability by a factor of around 4 times.

At site G2 considerable problems were encountered with premature gelling of the silicate grout due to high temperatures. Eventually all four primary holes were grouted with Geoseal MQ4. Permeabilities measured in the test hole showed variable results and, in retrospect, this was most likely due to the high test pressures being used causing hydraulic fracturing in the shallow holes. The testing of an intermediate hole S1 indicated completely impermeable conditions. This was interpreted as suggesting that the resin had been able to penetrate and seal the soils between primary holes.

At site G3 a large grout take was noted during initial backfilling of the tube-a-manchette. This was interpreted as soakage of the grout into coarse permeable features. All four holes were subsequently grouted to refusal using cement grout and normal cement grout techniques. Very modest grout takes were measured and testing of the central hole suggested that permeability had been reduced by only a factor of two.

In summary the testing programme was useful in showing that:

- Tubes-a-manchette could be effectively used in the site soil.
- Cement and silicate grouts could reduce permeability by a factor of 2 to 4.
- Silicate grout was difficult to handle in hot conditions due to accelerated gelling.
- Resin grout was easy to handle and there was some evidence that it could effectively seal the site soils.

7.1.5 Costs

Whilst the grouting effectiveness, particularly of cement grout, is not necessarily related to the volume injected it was convenient to consider the relative costs of various grouts on the basis of cost per litre. The costs of the materials as used in the test grouting programme are shown in Table 7.2.

Table 7.2
Cost of Grouting Materials

Grout Type	Material Cost per litre (1980)
Cement (10:1)	$0.01
Silicate (8:1:1)	$0.42
Resin (12.5%)	$0.33

Whilst each grout could be mixed and injected using equivalent conventional grouting equipment, both chemical grouts required special procedures for mixing in hot weather. The resin grout required mixing at a temperature less than 15°C although higher temperatures could be tolerated after the initial 15 minutes mixing period without reducing the gel time to an impractical level.

The silicate grout gel time was very dependent on temperature and even at moderate temperatures has a pot life of less than 90 minutes. This resulted in an increased injection cost over cement or resin due to continual interruptions to grouting with setting of grout in equipment.

7.1.6 Chemical Stability

Each of the three grout types were reported to be chemically stable under normal groundwater conditions, however, the possible effect of alkaline groundwater had to be considered.

Advice from manufacturers indicated that softening of the resin grout could occur under highly alkaline conditions but that the material is stable at the concentrations which could possibly develop at the Refinery Catchment Lake Dam.

On the other hand it appeared that silicate grout may be softened in relatively mild alkaline conditions and some testing would have been required prior to approval of its use.

Cement grout was considered resistant to alkaline conditions.

7.2 DESIGN DEVELOPMENT

From the results of the design phase grouting trials and literature search it was decided to specify a two grout grouting programme using firstly cement-bentonite grout and secondly Geoseal MQ4 resin grout.

A multi row grout curtain was proposed with an initial single row curtain of primary holes at 4m centres and compulsory secondary holes at 2m centres as an investigation phase. Pre-grouting permeability tests, of 2 metre soil stages, were required at alternate primary holes.

The initial grout results would be used to:

(i) develop a detailed understanding of soil permeabilities at the site, and

(ii) enable the determination of the extent of additional grouting required.

A tube a manchette system was proposed, and due to the lack of available information on practical grouting techniques for work of this type, the specifications were drawn up on a performance basis with the Contractor to be chosen with regard to expertise in the field. A detailed development programme in conjunction with the consulting engineers was specified as part of the grouting contract.

7.3 CONTRACTOR SELECTION

Tenders for the work were called on the basis of a Schedule of Rates Contract. Response from Contractors was poor, due probably to the limited expertise available in Australia for this type of specialist work.

A joint venture between a local Western Australian grouting firm, GFWA Pty Ltd and the French geotechnical firm S.I.F. Bachy Pty Ltd tendered an acceptable proposal and a contract was let for just over $2,000,000.

7.4 DEVELOPMENT OF GROUTING TECHNIQUES

7.4.1 Introduction

On site development work commenced in April 1981 with laboratory testing aimed at confirming manufacturers information about viscosity, setting, times, stability and effect of grout on site soils.

This work was followed up by practical field testing of alternative grouting techniques.

The work was carried out by the contractor's personnel in close liaison with the author and resulted in a report to the client recommending procedures to be adopted.

7.4.1 Laboratory Testing of Cement/Bentonite Grout

The specification anticipated the use of cement/bentonite grout for backfilling on installation of the tubes a manchette. This was specified to have a strength of 0.5 MPa at the time of grout injection and a strength of not more than 1.5 MPa at 90 days.

Laboratory testing enabled the preparation of data comparing the viscosity, stability rate of settlement of suspended particles and strength of various cement/bentonite/water mixes.

The results are presented in Table 7.3.

Neat cement/water grouts showed low stability and high set strengths with Marsh cone viscosity varying from 46 seconds for 0.8:1 mix to 29 seconds for 3:1 mix.

The addition of small amounts of bentonite to a cement/water grout was noted to result in a dramatic increase in stability. Viscosity was, however, significantly increased and strength was considerably reduced.

Mix No. 4 was judged to meet the requirements for backfill grout and was noted to have a viscosity similar to that of a 1:1 water/cement ratio grout.

Table 7.3 Cement Bentonite Grout

Mix No.	Unit	Cement	Bentonite	Water	W/C	Viscosity (sec)	Density	Anticipated Compressive Strength (kPa)		Measured Compressive Strength (kPa)		Decantation
								7 days	28 days	14 days		
1	kg	472	47	829	1.76	48	1.348	1500/ 1800	2200/ 2500	>500		<1 %
	l	295	47	829	2.81							
2	kg	429	42	845	1.97	42-44	1.316	1000/ 1500	1500/ 2000			<1 %
	l	268	42	845	3.15							
3	kg	393	39	858	2.18	39-40	1.290			>500		<1 %
	l	246	39	858	3.49							
4	kg	362	37	868	2.40	38-40	1.268			480		<1 %
	l	226	37	868	3.84							
5	kg	337	34	877	2.60	36-38	1.248	600/ 800	1000/ 1500	320		1.5 %
	l	211	34	877	4.16							
6	kg	314	31	886	2.82	35-37	1.232			330		1.5 %
	l	196	31	886	4.51							
7	kg	296	30	892	3.01	35-36	1.217			170		2 %
	l	185	30	892	4.82							
8	kg	276	28	900	3.26	35	1.205			140		2 %
	l	172	28	900	5.22							
9	kg	262	26	905	3.45	34-35	1.193	150/ 200	200/ 220	100		3 %
	l	164	26	905	5.52							
10	kg	1200	0	600	0.5	46	1.800	11000	21000	> 500		1 %
	l	750	0	600	0.8							
11	kg	1043	0	652	0.63	35	1.695	10000	17500	> 500		10 %
	l	652	0	652	1.0							
12	kg	632	0	789	1.25	29	1.421			> 500		36 %
	l	395	0	789	2.0							
13	kg	453	0	849	1.87	29	1.302			400 (1 day)		46 %
	l	283	0	849	3.0							

Up until the time of this laboratory testing, neat cement grout had been envisaged for the primary grouting phase. However the need to ensure that the tube a manchette valves were not 'locked up' due to the setting of a rigid grout zone around the valves following the cement grouting operation led to the consideration of an injection grout of similar nature to the backfill grout.

Deere (1982) discusses the consideration of cement/bentonite grouts and reasons that where grout set strength is not of importance then the benefits from a stable grout mix can outweigh the effects of increased viscosity.

The use of Mix No. 4 cement/bentonite grout was accepted for both backfill and injection grout.

7.4.3 Laboratory Testing of Chemical Grout

The specified chemical grout Geoseal MQ4 is a phenoplastic resin, soluble in water. It is supplied in a dry powder form in preweighed 24kg bags with 2 kg sachets of the catalyst, flake caustic soda enclosed.

The laboratory programme specified:

(i) development of mixing techniques

(ii) field confirmation of the manufacturers data for viscosity/time relationship

(iii) confirmation of grout performance in alkaline conditions

(iv) evaluation of grout penetrability into foundation soils.

These items are discussed below:

(i) Mixing

Borden Chemicals' recommended mixing procedure was to first mix the caustic soda with the required volume of water and then to add and mix the resin powder.

Simple tests on a laboratory sample determined the following information:

- time for complete dissolving of caustic soda was 3 minutes.

- temperature rise during caustic soda mixing was up to 6 C. ^{o}

- time for complete dissolving of resin powder was 15 minutes.

- temperature rise during resin mixing was 2^{o}C.

(ii) Viscosity

Testing for viscosity relationships included comparison of results for grout mixes with:

a) slightly varying concentration of resin and catalyst
b) mixing temperature
c) curing temperature

Viscosity was measured using a Baroid Rheometer which is a rotational viscometer giving readings in Centipoise. A comparison of various viscometers is presented in Appendix E.

The results are shown graphically in Fig 7.4 and 7.5 and summarised below:

The manufacturers information on the effects of temperature and concentration on setting times was generally confirmed.

Increased grout concentration leads to increased viscosity. A 12.5% solution was adopted to give a low initial viscosity.

A slight variation in catalyst ratios had no observable effect on the viscosity/time relationship.

Viscosities of 3 to 5 centipoise were consistently recorded. This compares with the manufacturers claims of an initial viscosity of 2 centipoise. Some variation in viscometers and accuracy may partially explain the result but most likely manufacturers claims are optimistic.

For the purpose of the project the grout was considered gelled when a viscosity of 6 centipoise was reached.

At this viscosity the effective penetration ability of the grout was greatly reduced over that of fresh grout.

It was found that gel time decreased with higher curing temperatures and was also shortened by higher mixing temperatures. This fact became particularly significant in later field trials resulting in the need to pre-cool the mix water with ice.

(iii) Stability

Testing of the grout stability proved difficult with the limited site laboratory facilities. Some success was achieved by placing grout samples in various solutions, intended to represent future groundwater properties, and by analysing the change in solution properties with time. The results are summarised in Table 7.4.

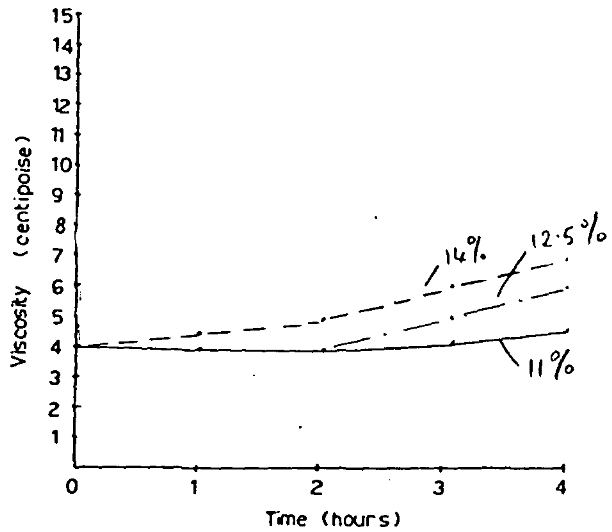


Figure 7.4. Resin Grout
Effect of Concentration

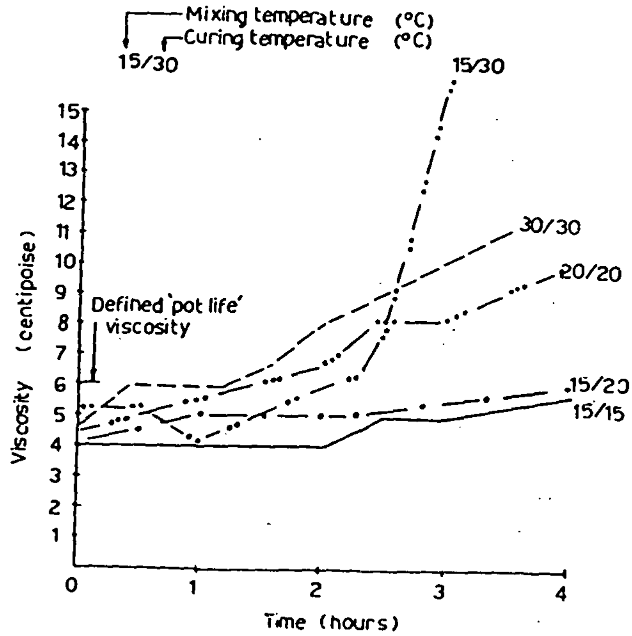


Figure 7.5. Resin Grout
Effect of Temperature

TABLE 7.4

GROUT STABILITY TESTING

Sample No.	Solution	Properties measured after 10 days			
		Phenols (mg/l)	pH	Colour (platinum-cobolt Scale)	T.D.S.
1	water (pH7)	0.15	7.9	2800	2600
2	NaOH;4mg/l(pH9)	.1	7.5	2000	2000
3	NaOH;40mg/l(pH11)	.1	8.2	2800	2300
4	Na_2CO_3;40mg/l(pH10)	.1	7.6	2200	2100
5	Na_2CO_3;400mg/l(pH12)	.1	9.3	3600	2500
6	water without grout	.1	7.5	15	210

The concentration of phenols was highest in the water solution with levels in saline solutions being lower than the level of detection by the test method.

The pH readings of the saline solutions were noted to decrease. This was thought to be at least partially due to the dissolving of CO_2 from the atmosphere.

No positive variations could be detected in the colour or total dissolved solids (T.D.S.) results apart from the higher colour reading of the concentrated sodium carbonate solution which represents a far harsher environment that that expected to develop beneath the dam.

The colour and TDS results showed a clear tendency for some breakdown of the gel in solution, although the tendency was, if anything, lower in alkaline solutions than in water

These results were seen as confirmed the manufacturers advice that the grout was stable in the range of alkaline conditions expected at the site.

(iv) Evaluations of Penetrability into Site Soils

A limited amount of laboratory testing was carried out to investigate the effect of grout penetration in soil samples. The results were less than satisfactory but lack of time and interest on behalf of the Contractor prevented the continuation of the work.

In one set of tests, samples of varying permeability were fabricated from on site soils and placed in PVC tubes 50 millimetres diameter and 1 metre long. Water was first passed through the samples to measure permeability and then grout was injected at similar pressures.

Permeabilities ranged from 4×10^{-5}m/s to 1×10^{-2}m/s.

Flow could be maintained using resin on only one sample, being the most permeable. The apparent permeability indicated was 2×10^{-7} m/sec representing an apparent viscosity of approximately 5×10^{-4} centipoise.

Similar tests were carried out on samples of finer soil only 100 millimetres in length and permeabilities between 2×10^{-7} and 4×10^{-8} metres per second.

No flow of resin through the samples could be generated and application of progressively higher pressures eventually led to failure of the samples by flow of resin between the samples and the tube walls.

Inspection of the samples showed resin penetration of only 3 millimetres.

Whilst not conclusive, these results indicate that even a very low viscosity grout does not readily penetrate low permeability soils and that the mechanism of grout flow in soil involves a more complex relationship than that of water flow in similar soils.

Further investigation of this problem was beyond the scope of the laboratory testing programme and of this thesis.

7.4.4 Tube a Manchette Design Development

The specified tubes a manchette were 32 millimetre diameter PVC Class 9 pipes with rubber 'valves' at 300 millimetre centres. At the Refinery Catchment Lake Dam the tubes a manchette were specified for rock grouting as well as for soil grouting. This was an unusual application as in conventional rock grouting great care is taken to wash the holes to expose clean joints, fractures or other permeable features prior to grouting.

Rock grouting was initially specified to be carried out at pressures of 50 D kPa where D represented the depth in metres from the ground surface to the point of grout application. Furthermore, early lack of understanding of the low actual pressure of valve opening, as discussed later, led to quite high grout pressures being applied.

During early testing it was found that the Class 9 pipes were being deformed by the pressures involved and difficulties were experienced with relocation of the inner grout tube. This led to the substitution by 40 millimetre diameter Class 18 pipe.

Field testing and subsequent production grouting led to the confirmation that the rock foundation could be effectively grouted by the tube a manchette method with obvious reduction in grout take in successive grout holes in a closure pattern. It was postulated that the pressures used were sufficient to fracture the backfill grout, separate it from the borehole wall and thus penetrate the rock features.

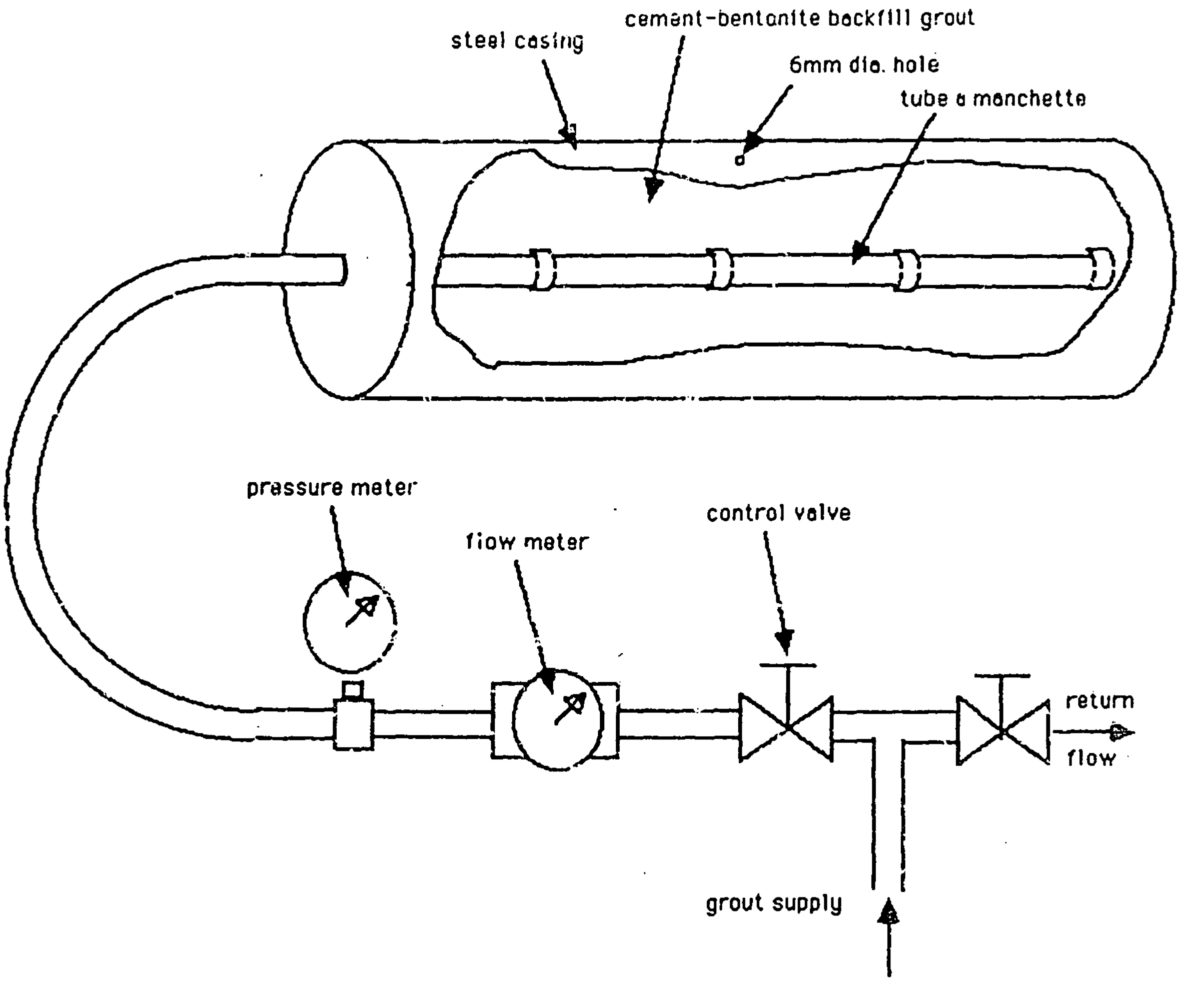


Figure 7.6 Tube a Manchette Trial Section

Another unusual aspect of the specified grouting technique was the use of an inner grout pipe system which enabled the pressurisation of three tube a manchette valves simultaneously.

Several tests were carried out, using tubes a manchette sections cast in steel and PVC pipe sections which conclusively demonstrated that all three valves opened at very low internal pressures. A typical test apparatus is shown on Fig. 7.6.

7.4.5 Grout Pressures

The project specification, developed from a review of literature none of which referred specifically to soils of as low a permeability as those at the Worsley site, required grout pressures of:

(i) in rock - 50 D KPa, and
(ii) in soil - 5 D KPa rising to "ground fracture" pressure,

where
D denotes the depth of the stage being grouted, in metres, and,
'ground fracture' pressure was defined as the pressure at which a large increase in flow rate was observed for a small increase in applied pressure.

In both cases these pressures were to be added to the "valve opening" pressure which was conceived as the head loss across the rubber tube a manchette valve.

The Contractors introduced a further concept of "valve bursting" pressure which was defined as the pressure to initially 'crack' the rubber valve open.

The expected flow/pressure relationship for soil is shown in Fig. 7.7.

Initial field trials by the Contractor reported very high valve opening pressures, in the order of 100 D KPa. The Contractor was not alarmed by these values and claimed them to be normal. A section of grout curtain was constructed using these high pressures and tested.

The test comprised drilling and grouting of a four metre length of curtain comprising two primary holes 4 metres apart, a secondary hole between these two and a water test hole between the first primary and the secondary hole.

Despite several technical problems with the tubes a manchette as discussed previously and with the water testing technique to be discussed later, several important observations were made.

Water testing during drilling was carried out over 2 metre stages initially using an inflatable rubber packer at the top of the stage and a test pressure of 5D.

Following tube a manchette installation some further water testing was carried out through the tube a manchette to attempt a correlation with the open hole tests.

Permeability test results are summarised in Table 7.5.

TABLE 7.5

Effect of Test Grouting on Permeability

Depth (m)	Permeability (m/sec) Pre-grouting	Post grouting
4 - 6		1.3×10^{-5}
6 - 8	10^{-8}	9.1×10^{-8}
8 - 10	10^{-8}	5.9×10^{-8}
9 - 12	1.4×10^{-7}	2.3×10^{-7}
12 - 14	2.7×10^{-7}	1.7×10^{-8}
12 - 16	3.7×10^{-7}	1.8×10^{-6}
15 - 18	1.4×10^{-7}	1.9×10^{-6}
15 - 20	10^{-6}	1.7×10^{-6}

As can be seen the final permeability tests showed no improvement over the original tests with an apparent increase in permeability by up to a factor of 10 where initial permeabilities were lowest.

The results prompted a close scrutiny of the Contractors techniques.

As is usual in field grouting the injection pressure was controlled by operator adjustment of diaphragm valves on the grout line and a recirculation line. A pressure gauge is located on the downstream side of the grout line valve.

For water testing and resin grouting a water meter was located at the gauge. The meters used were standard municipal type domestic water meters calibrated in litres.

The procedure was for pressure to be increased to "valve burst" pressure at which flow was first noted. Once flow had been established the pressure was dropped until flow ceased, whereupon it was increased again to register flow, usually at a much lower pressure than "burst" pressure.

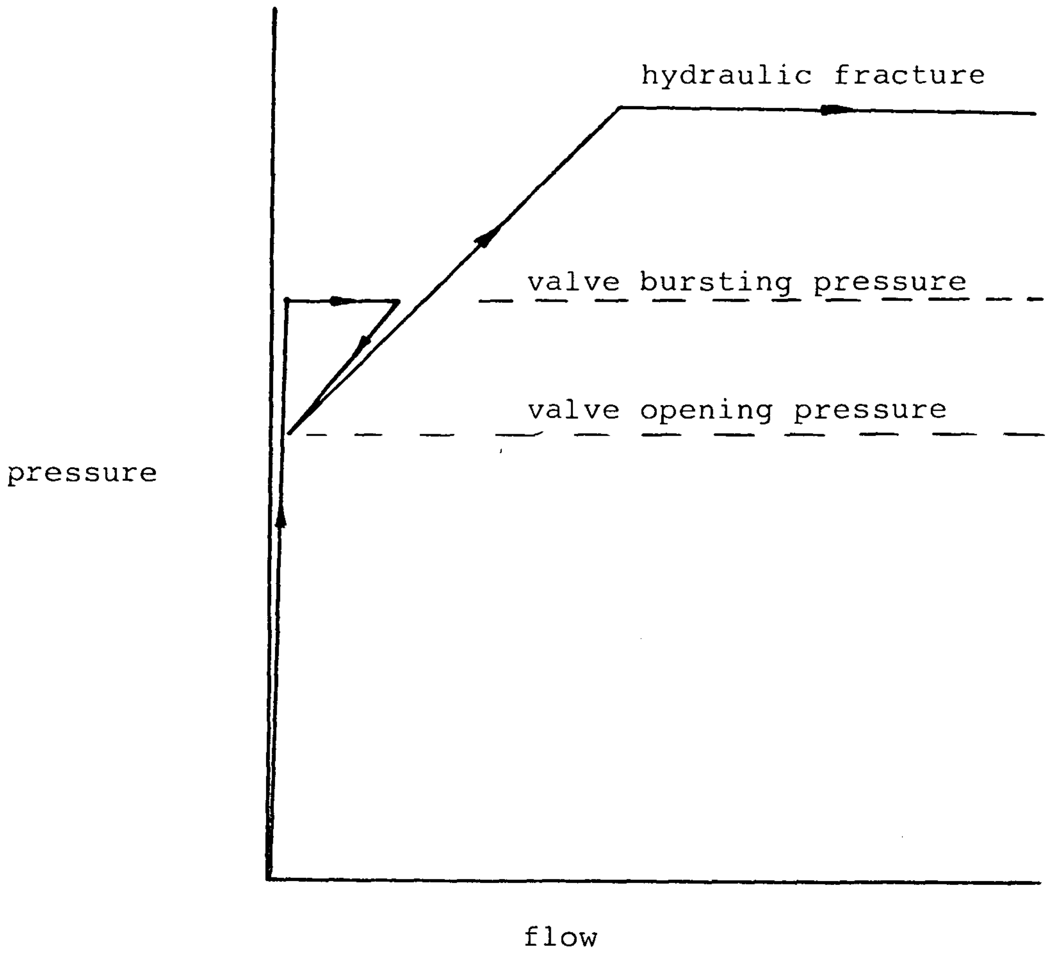


Figure 7.7. Expected Grout Pressure/Flow Relationship

This pressure was called "valve opening" pressure. The required grout pressure was added to this valve opening pressure.

Grouting records indicated that 'valve bursting' pressure regularly exceeded 1000 kPa and sometimes reached 2000 kPa which was close to the capacity of the injection equipment.

It was suspected that excessive pressures were being used due to:

(i) low sensitivity of meters

(ii) lack of operator experience and appreciation of the work.

This led to laboratory testing of tubes a manchette valve opening and field testing using more accurate flow gauges under more highly supervised conditions.

The results are described below:

(a) Laboratory Testing of Valve Opening

Sections of tube a manchette were cast into PVC and steel pipe sections as shown in Fig. 7.6. Flow was established through the "tell tale" holes in each test at pressures of between ten and forty kPa.

(b) Field Testing

A water meter measuring to 0.1 litres was used to test opening pressures and grout rates of hole No. AP200 using first water and then resin. The results were very different from previous work.

'Valve Bursting' pressure and 'valve opening' pressures were found to be virtually identical and quite low, ranging from 2D to 3D for water and from 3D to 10D for resin.

Results are presented in Table 7.6.

TABLE 7.6

Tube a Manchette Valve Opening Tests

Hole No. AP200

Stage (m)	Bursting Pressure		Opening Pressure		Grout Pressure		Flow Rate	
	Water	Resin	Water	Resin	Water	Resin	Water	Resin
6.7-7.6	20	60	18	55	53	85	3.6	1.7
18.4-19.3	40	70	13	65	120	155	4.4	5.5

Flow rates of water and resin at similar pressures were also noted to be consistent with expectations from viscosity theory, in contradiction to laboratory testing previously described.

From theoretical work on ground fracture described in Chapter 4 it is apparent that hydraulic fracture of the Worsley soils could develop from pressures as low as 10D. Thus the previously used 'valve bursting' and grouting pressures of up to 100D were likely to have been causing quite extensive fracturing and distribution of grout over large distances. Subsequent excavation in grouted areas revealed evidence of this effect. Most often in the form of a single sub vertical fracture of several millimetres width extending several metres in random directions.

Data collected during the development phase indicated:

(a) Tube a manchette valves opened at low pressures.

(b) Permeabilities over the site were generally low.

(c) Ground fracture of initially low permeability soil was not effective in reducing permeability and could in fact be detrimental.

Accordingly, the Contractors were instructed to reduce grouting pressures to 15D for overburden and 30D for rock. In addition, valve bursting and opening pressures were not considered further, resulting in a far more simplified field procedure for the operators.

Later, during production grouting, continual ground fracture was noted when rock grouting, particularly beneath deep overburden. Fracturing was inferred by large grout takes out of context with measured permeabilities. Surface leakage was observed in several cases.

The fracturing appeared due to the exaggeration of the rock grouting pressure by the depth of overburden. Hence when grouting a 5 metre stage of rock beneath 30 metres of overburden, the grout pressure of 900 KPa is actually 180D relative to the rock alone.

The rock grouting pressure was subsequently also reduced to 15D.

7.4.6 <u>Grout Injection Volumes</u>

With the reduction in pressures and the decision not to purposely initiate ground fracture, the usual tube a manchette grouting procedure of injecting a fixed volume of grout was not appropriate.

Instead, cement/bentonite grouting was carried out to refusal in accordance with conventional grouting techniques, with refusal defined at a flow less than 3 litres/15 minutes, whilst resin grouting was carried out for a period of 300/D minutes.

This time period was chosen as sufficient to enable effective grouting to a radius if 1.25 metres (slightly greater than the hole spacing) of material with a permeability of 10^{-5}m/sec. Hence grouting at a depth of 20 metres would be carried out for 15 minutes. This grout/time allowance was intended to result in zones of similar permeability at different depths, being grouted with the same volume of grout. That is, soil at depth would be grouted at a higher pressure for a shorter time and vice versa.

An upper limit to grout take at any stage was set at 1000 litres.

Refusal flow for resin grout was set at 1 litre per metre per 100 KPa which was calculated as the approximate grout take to a stage with a permeability of 10^{-6}m/sec.

7.4.7 Stage Length

The selection of the number of tube a manchette valves to pressurise at one time was somewhat controversial, with the Contractor claiming that only one valve at time should be used. Field trials, however, suggested that acceptable results were obtained by pressurising three valves simultaneously. Additional costs in the order of $1,200,000 would have been incurred if each valve had been pressurised independently.

7.4.8 Grout Type Allocation

The initial grouting procedures involved cement/bentonite grouting of all stages, followed by chemical grouting of all stages. It quickly became apparent that:

a) Cement/bentonite takes in soil were negligible.

b) Resin takes in rock, following cement/bentonite grouting, were negligible.

This led to deletion of cement/bentonite grouting in all but the lower two stages of the soil and the deletion of resin grouting in rock. This considerably reduced the number of hook-ups required, which more than compensated for the cost of possible increased resin takes in the soil.

7.4.9 Water Testing

Water testing was carried out over two metre stages in open boreholes, initially at a pressure of 5D KPa using inflatable rubber packers above the stages being tested.

Continual problems, with water leaking past packers and with packers bursting due to over inflating, led to the abandonment of this type of test in soil.

A simple falling head test was developed, as shown in Fig. 7.8, using low head in open drill casing. The casing was progressively advanced past the previously drilled level, without using water recirculation. This bedded the casing into the soil. A two metre hole was then drilled past the end of the casing, and the casing filled with water. Leakage past the casing seal was noted in approximately 5% of the tests; however, results were generally satisfactory. The effect of any leakage past the seal would lead to overestimating of permeabilities.

4.4.10 Gamma Logging

As a possible means of pre-determining the areas of higher permeability, the use of gamma logging was attempted. This involves the lowering down a borehole of a device which measures the level of gamma radiation emitted by the soil.

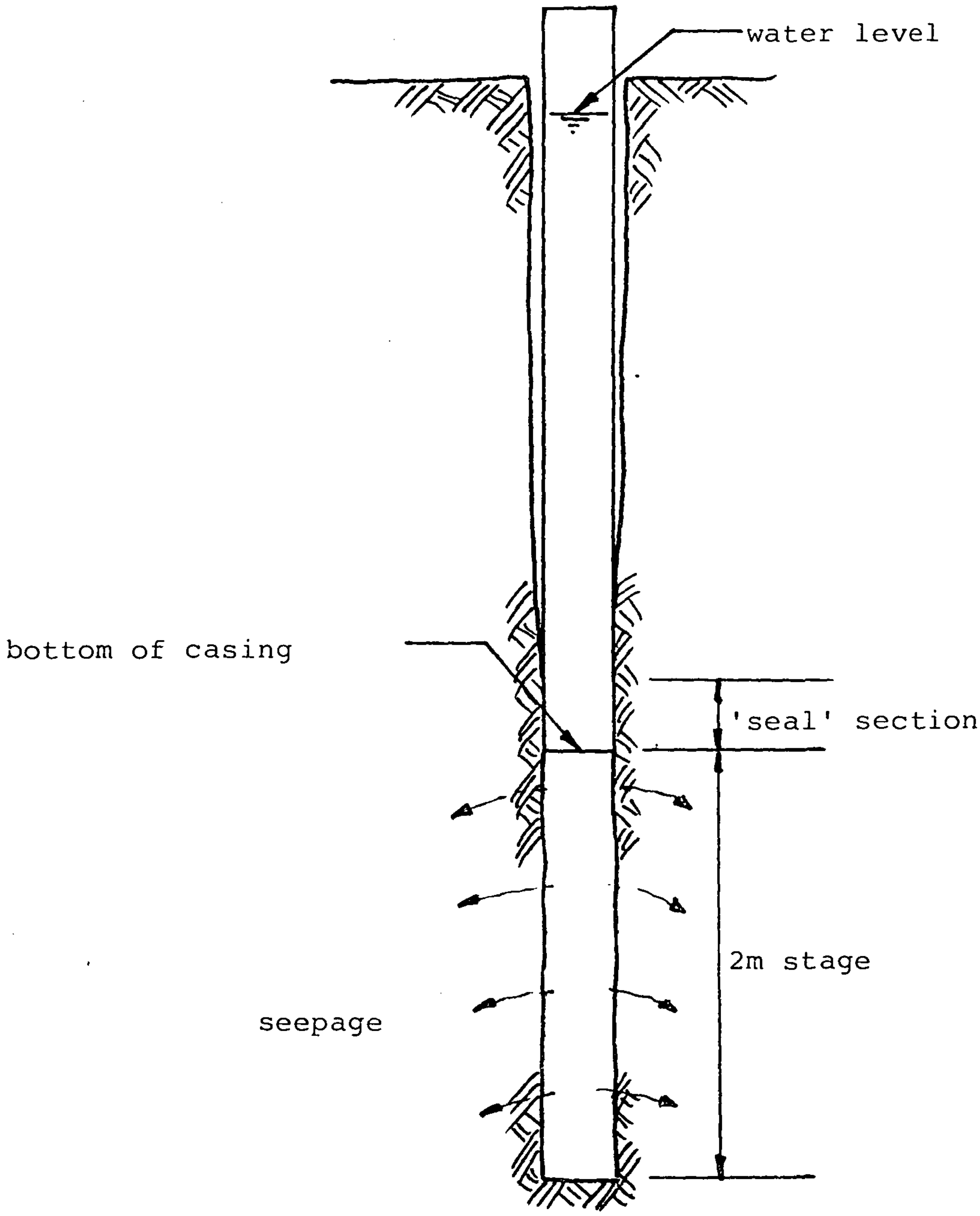


Figure 7.8. Falling Head Permeability Test

The level of radiation from sands is markedly different to that from clays and it was considered that zones of low radiation may indicate areas of higher permeability.

Whilst early work in this regard suggested that some correlation may exist, it could not be established that the results would be sufficiently conclusive to enable confident interpretations to be made.

7.5 CONSTRUCTION PHASE

7.5.1 Construction Programme

The very tight refinery construction programme and very wet winter weather of the region necessitated the construction of the main embankment for the Refinery Catchment Lake Dam during the summer of 1981/82. This meant that the grouting operation was completed during the winter and spring of 1981. The greatest part of the works was completed during an eighteen (18) week period between 15th August, and 22nd December 1981.

7.5.2 Production Statistics

Final extent of work on Refinery Catchment Lake Dam was as follows:

Drilling overburden 23,300 metres
Drilling rock 3,427 metres
Grout hook ups 25,640 No.
Permeability tests 2,485 No.
Cement 207 tonnes
Bentonite 39 tonnes
Resin grout 123 tonnes

To complete this extent of work in the time available required double shift operation. Work was arranged on the basis of ten 12 hour shifts per week. A total, on site input of 82,000 man hours was involved in the operation.

7.5.3 Construction Techniques

The use of the Tube a Manchette technique made it possible to allow the drilling operation to proceed largely independently of the grout works allowing construction flexibility. Various construction methods were tried or considered in an attempt to optimise production. The most successful methods developed may be described as follows:

Production Drilling - a truck mounted Edson rotary top drive rig was used to drill overburden and place temporary steel casing to rock. Conventional air track rigs were then used to drill rock and, after placement of Tubes-a-Manchette, withdraw casing.

Permeability Test Drilling - because permeability tests were necessary in 2 metres stages (downstage) through the overburden the actual water test represented the bulk of the time spent on this operation. For this reason a drill rig which could be moved between holes during the testing was necessary. To reduce the set up time involved, a specially modified air track with rotary head was used for the bulk of this work.

Cement Grout Mixing and Pumping - most cement/bentonite grout was mixed in a central batching plant and pumped via steel lines to mobile holding tank and pump units. All pumping was carried out with Mono Pumps.

Resin Mixing and Pumping - resing grout was mixed in a central batching plant and delivered along the grout curtain line by a pressurised reticulation system, whereby the grout was available at maximum injection pressure at any required location. Grout flow was tapped from the pressure line to a manifold system from which individual hole connections were made. Return lines, from each hole, connected via a main return line to the central batching plant.

In this way a large number of individual hole connections could only be made concurrently from one pump. Generally, it was possible to grout on up to ten holes simultaneously.

Control of grout temperature and monitoring of viscosity avoided the possibility of 'gelled' grout being recirculated. Grout in excess of 1 hour old was wasted.

7.6 RESULTS

A total of 896 pre-grouting and 1438 post grouting permeability tests were completed at the RCL Dam Site. The results of these are shown in Fig. 7.9 and summarised in Fig. 7.10.

In general, natural foundation permeabilities were very low with the highest recorded permeability being 1 x 10^{-4} metres per second. Less than 10% were higher than 1 x 10^{-5} metres per second.

Furthermore, no distinct pattern could be detected in the distribution of higher permeability zones. It was thus determined that whilst the seepage analyses carried out at the design stage using models, including sand lenses, were realistic at isolated sections, the permeable features were not continuous and the overall seepages calculated were likely to be overestimates.

Permeabilities were however, noted to be, on average, slightly higher over the central section of the valley. Consequently this led to the installation of an additional two rows of grouting over the central 200 metres of the curtain whilst the remaining, less critical areas were limited to one row.

Final permeability test results showed that the extent of more permeable foundation areas had been dramatically reduced and the overall average permeability had been reduced by a factor of around five times.

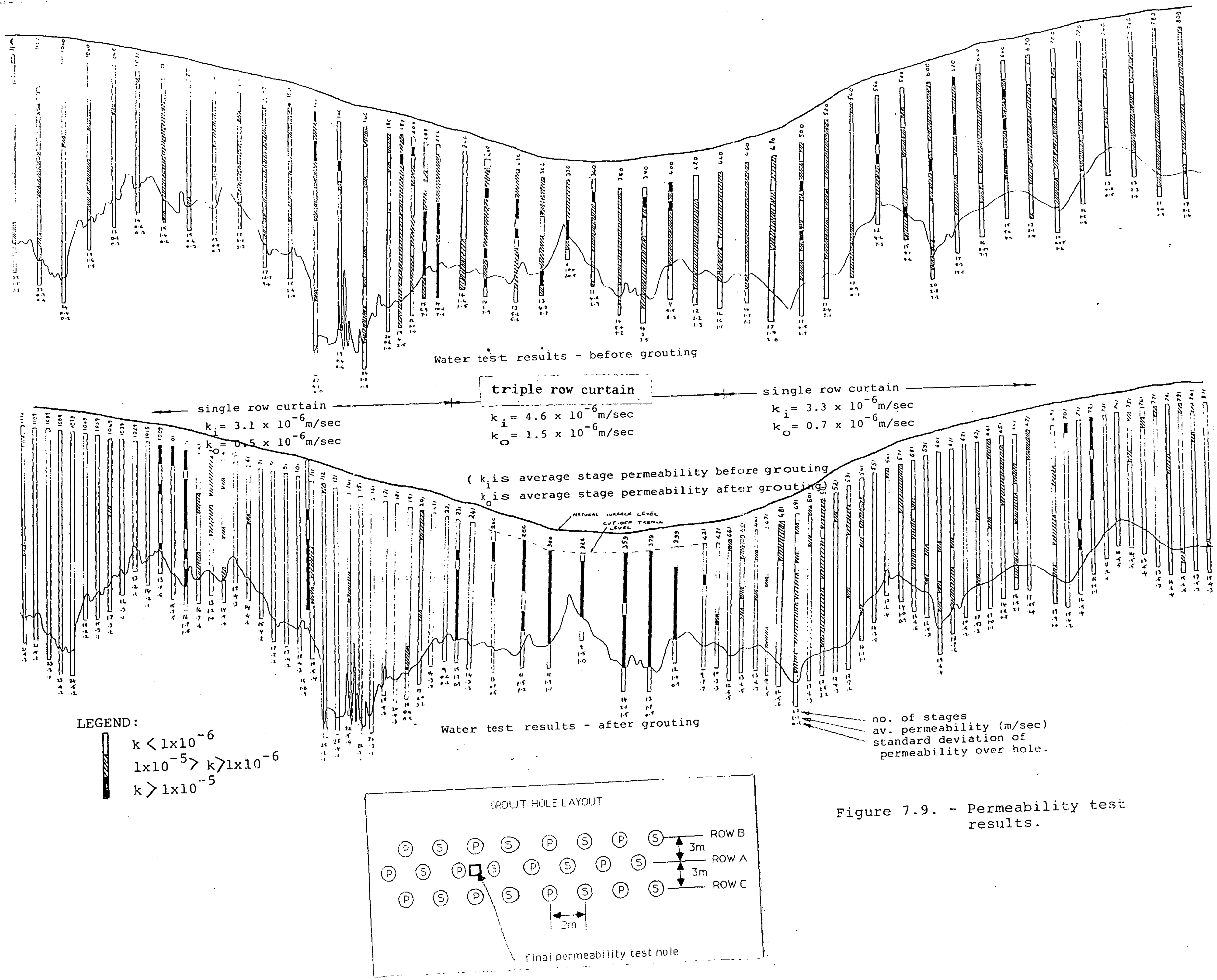


Figure 7.9. - Permeability test results.

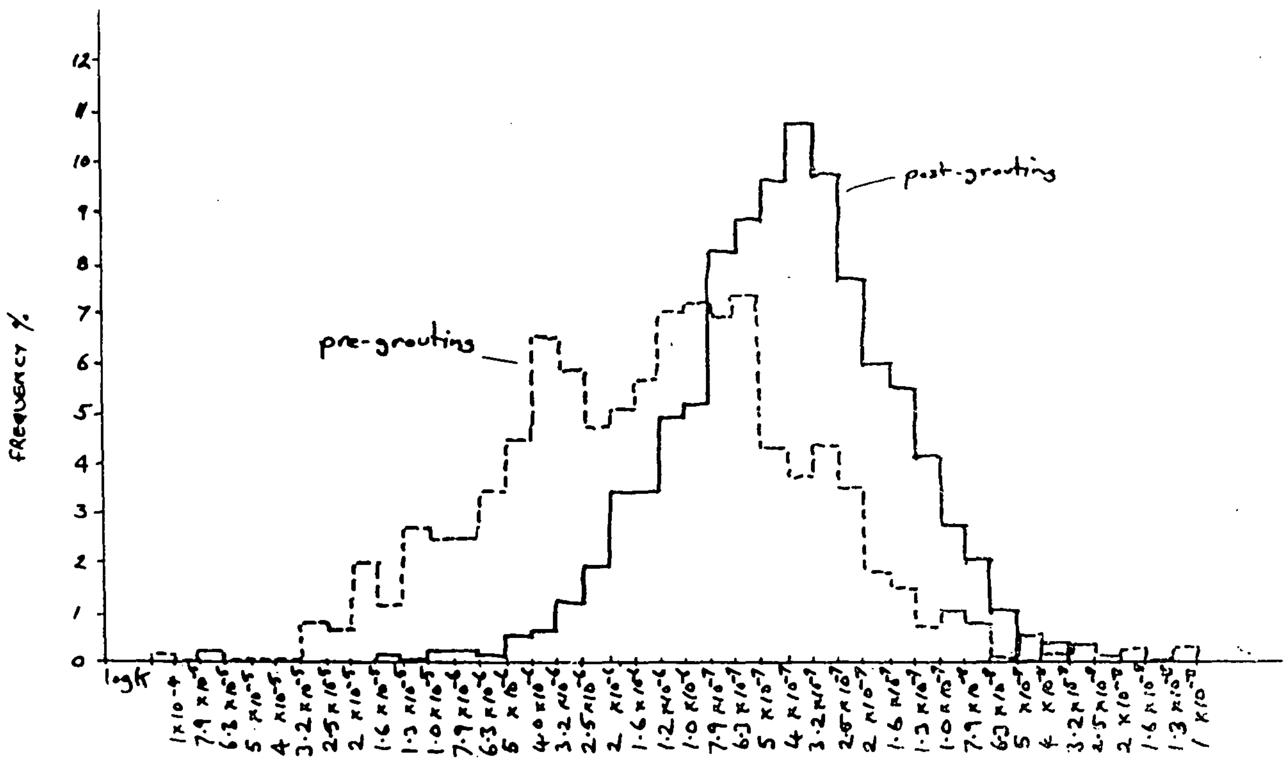


Figure 7.10. Frequency of permeability test results.

Due to the inherent inaccuracy in the permeability test method used, particularly when attempting to measure post-grouting permeabilities in a single row curtain, these results are considered conservative and true post-grouting permeabilities are likely to be even lower.

7.7 POST GROUTING PERFORMANCE

Filling of the reservoir began in May 1982, and over 15 metres of water were stored by August 1983.

The level has remained virtually constant since then. Instruments in the dam and foundations are being monitored and chemical analysis of water downstream of the dam is being carried out routinely.

Reports from the site up to May 1985 indicate no measurable rise in water table levels downstream of the dam and no evidence of seepage.

CHAPTER 8 - SUMMARY AND CONCLUSIONS

8.1 SUMMARY

This thesis has described the development of part of an engineered 'water management system' design to eliminate any risk of environmental damage from the construction and operation of the Worsley Alumina Refinery.

The construction of a grout curtain beneath the Refinery Catchment Lake Dam, the ultimate storage reservoir of water contaminated with caustic soda and other salts from the refinery process, required the development of unique grouting techniques in the already low permeability silty clay soils.

Seepage analyses showed that if certain postulated permeable zones existed beneath the dam, then significant volumes of contaminated seepage could by-pass other protective devices downstream of the dam. A major programme of drilling, testing and grouting was proposed to confirm or disprove the presence of permeable zones, and if present to seal them off.

Grouting of soil foundations is a relatively modern practice. The first documented use of grout was by the French Engineer Charles Berigny in the early 19th century.

It was not until 1925, however, that soil grouting was considered feasible with the introduction of silicate grout by H J Joosten.

Since that time development has been rapid with the Tube-a-Manchette being introduced by Ischy in 1933 and the range of available grouts expanding dramatically in recent years.

Grouts can be considered as falling into three groups, namely, suspensions, colloidal solutions and pure solutions.

The later group includes acrylamides, aminoplasts and phenoplasts, some of which can be produced with viscosity close to that of water enabling penetration into extremely fine voids.

Despite the fact that pure solution grouts behave very much as Newtonian fluids, the physics of grout behaviour is complex with many factors involved, often being difficult to precisely define. These factors include soil grain size, grain shape, chemical properties and grout pressure effects.

Of major significance is the effect that the grouting process can have on the soil body as a whole. Increased pore pressure during grouting can exceed the tensile strength of the soil and cause hydraulic fracturing. Testing at Worsley showed that this could occur at pressures significantly below overburden pressure.

Hydraulic fracturing is often exploited in soil grouting operations to enable more rapid injection of grout.

At Worsley, a dual grout system comprising cement grout and a phenoplast grout, Geoseal MQ4, was proposed.

The desirability of using ground fracture techniques and intentionally introducing cracks into already low permeability soils was questioned by the Author. Vaughan (1971) had observed that, once created, fractures re-opened at lower pressures than the initial fracture pressure. Chemical grout gels are weak when not incorporated in a soil mass and even the lowest viscosity grout could have negligible penetration into soils with permeabilities less than 10^{-7} metres per second over the relatively short "live" period prior to gelling. Thus a band of weak gel injected into a fracture in otherwise, effectively impermeable soil, was unlikely to be effective in further reducing permeability and could form a weakness in the soil mass.

Due to the uniqueness of the proposed works a contractor was chosen on the basis of past experience and the initial phase of the work was the development, in conjunction with the Author, of suitable techniques.

Laboratory and field testing suggested that grouting procedures used by the contractors on past projects, in more permeable soils, were not effective at Worsley. Whilst testing was limited, evidence suggested that grout pressures were excessive and permeability may have been increased by the grouting process.

A low pressure grouting technique was developed which was practical and enabled the contractor to complete the work within the project budget and time schedule.

Extensive permeability testing prior to and during grouting did reveal some permeable features but these appeared to be discontinuous and not as severe as some of the more pessimistic design stage assumptions.

Furthermore, testing on completion of the work showed a positive reduction in permeability over the curtain to the extent that the small amount of seepage that may occur will be safely collected by the downstream collection systems incorporated in the design.

8.2 CONCLUSIONS

It is likely that techniques for grouting low permeability soils will continue to develop with increasing technological advances and understanding. It is also highly likely that many factors, particularly desirable grout pressures, will be controversial items.

The experience at the Refinery Catchment Lake Dam has shown that low permeability soils can be successfully grouted, but that techniques developed for alluvial grouting may not be appropriate.

In particular the high pressure injection of a fixed volume of grout at each stage must be questioned.

An alternative technique using low pressures proved practical in the field and achieved positive results.

It is concluded that the design aim of the grouting programme at the Refinery Catchment Lake Dam was achieved. The final cost was in the order of $2,500,000.

8.3 FURTHER WORK

With increased awareness of environmental matters together with the need for the development of increasingly less favourable dam sites, it is likely that chemical grouting of low permeability foundations will become more common.

The work on this project has highlighted several areas where further work would be of benefit.

The first of these is with regard to the effect of the creation of a grout filled seam, or band, in a low permeability soil mass.

Work by Jaworski et all (1981), Bjerrum et al (1972) and Enever et al (1976) has been previously described. These researchers investigated the development of hydraulic fractures in laboratory samples.

It is proposed that similar testing using grout would be useful to determine the extent of the weakness introduced by the non penetrating grout gel.

The work might take the form of encasing a grout source in earth samples of various permeabilities, injecting grout under pressure to cause fracture and testing permeability after grouting by injecting water under gradually increased pressure.

This would determine if the grout seam in lower permeability soils behaved in a similar manner to a fracture caused initially by water, and thus enabled the creation of the seepage path along the fracture by compression of the gel.

A second source of research work could result from the great number of permeability tests made during the construction work. These would provide the opportunity for a detailed analysis of seepage predicted from a three dimensional seepage model.

Correlation of seepage predictions with measured seepage from dam monitoring could enable a check on the theoretical model.

Theoretical analysis of the measured permeabilities would at least make an interesting review of the actual need for construction of the grout curtain and may be valuable as a guide to the degree of foundation treatment required for other projects in similar geological conditions.

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APPENDICES

Appendix A - Submission for 1983 I.E.(Aust). Engineering
Excellence Award.

THE INSTITUTION OF ENGINEERS,
AUSTRALIA

BE IT KNOWN

THAT THE

WESTERN AUSTRALIA ENGINEERING AWARD

1983

HAS BEEN MADE TO

GHD – DWYER (WA) PTY. LTD.

FOR

WORSLEY ALUMINA PROJECT
WATER MANAGEMENT

IN RECOGNITION OF DISTINCTION
IN ENGINEERING ACHIEVEMENT

PRESIDENT

CHAIRMAN
W.A. DIVISION

ASSOCIATION OF CONSULTING ENGINEERS AUSTRALIA
WESTERN AUSTRALIAN CHAPTER

ENGINEERING MERIT AWARD

be it known that

WORSLEY ALUMINA PROJECT - WATER MANAGEMENT

designed by

GHD - DWYER (W.A.) PTY LTD

has been awarded

ENGINEERING MERIT AWARD 1984

GIVEN IN RECOGNITION OF AN ENGINEERING PROJECT OF OUTSTANDING MERIT, SIGNIFICANT BENEFIT TO THE COMMUNITY AND TO THE FURTHERMENT OF CONSULTING ENGINEERING.

Peter A. Leadbetter

CHAIRMAN OF THE W.A. CHAPTER ACEA.

J. Buchanan

CONVENOR OF PANEL OF JUDGES.

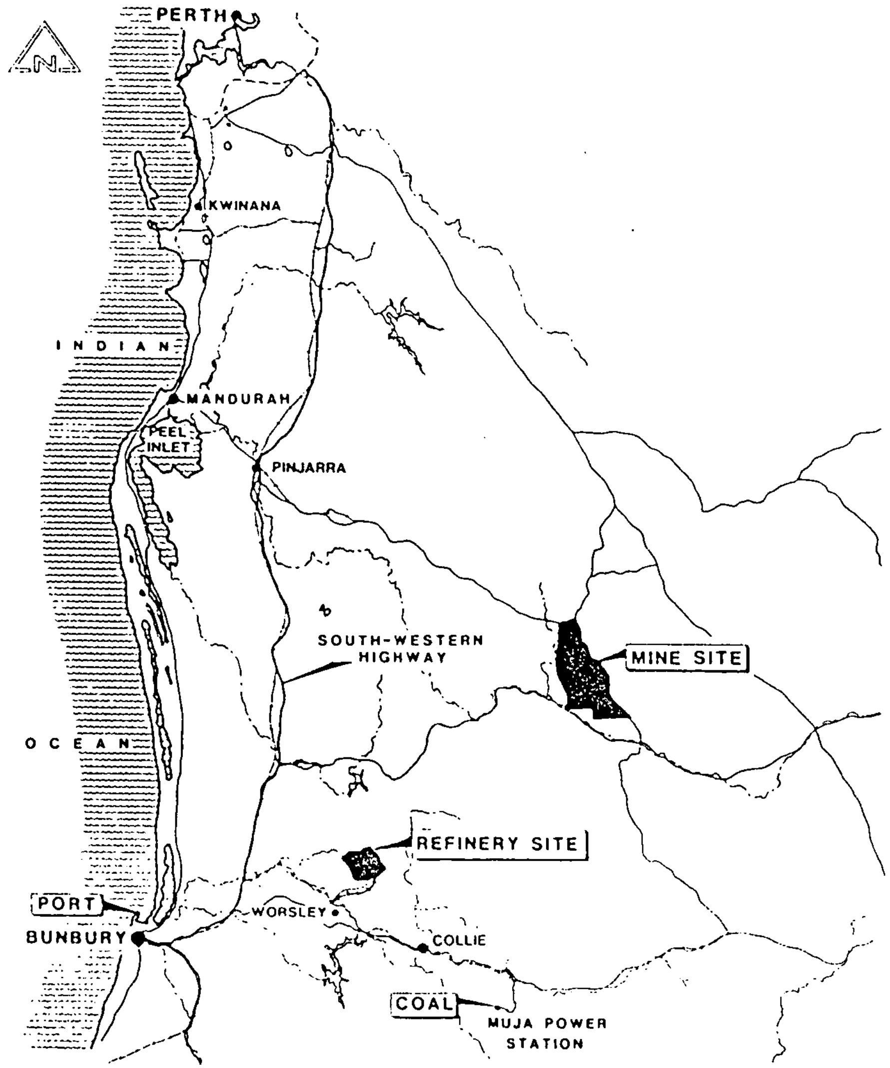
The Institution of Engineers, Australia

WESTERN AUSTRALIAN DIVISION
ENGINEERING AWARDS
1984

WORSLEY WATER MANAGEMENT

GHD

GHD-Dwyer (WA) Pty Ltd
Consulting Engineers Project Managers
7 Hardy Street South Perth 6151



SELECTION OF SITE

An alumina refinery presents a challenge in any environment since the processing of bauxite leaves huge quantities of residue which typically contain significant quantities of caustic compounds. These residues, plus caustic spillages that might occur within the refinery and other wastes such as cleaning acids need to be isolated from the environment.

Due to the large quantities of residue in the ore, considerations of haulage would place the refinery as near as possible to the mine - in this case on the western slopes of the Darling Range where water supplies are of poor quality. An alternative site near the major port of Bunbury was rejected because of its proximity to a fast growing community and poor foundation conditions.

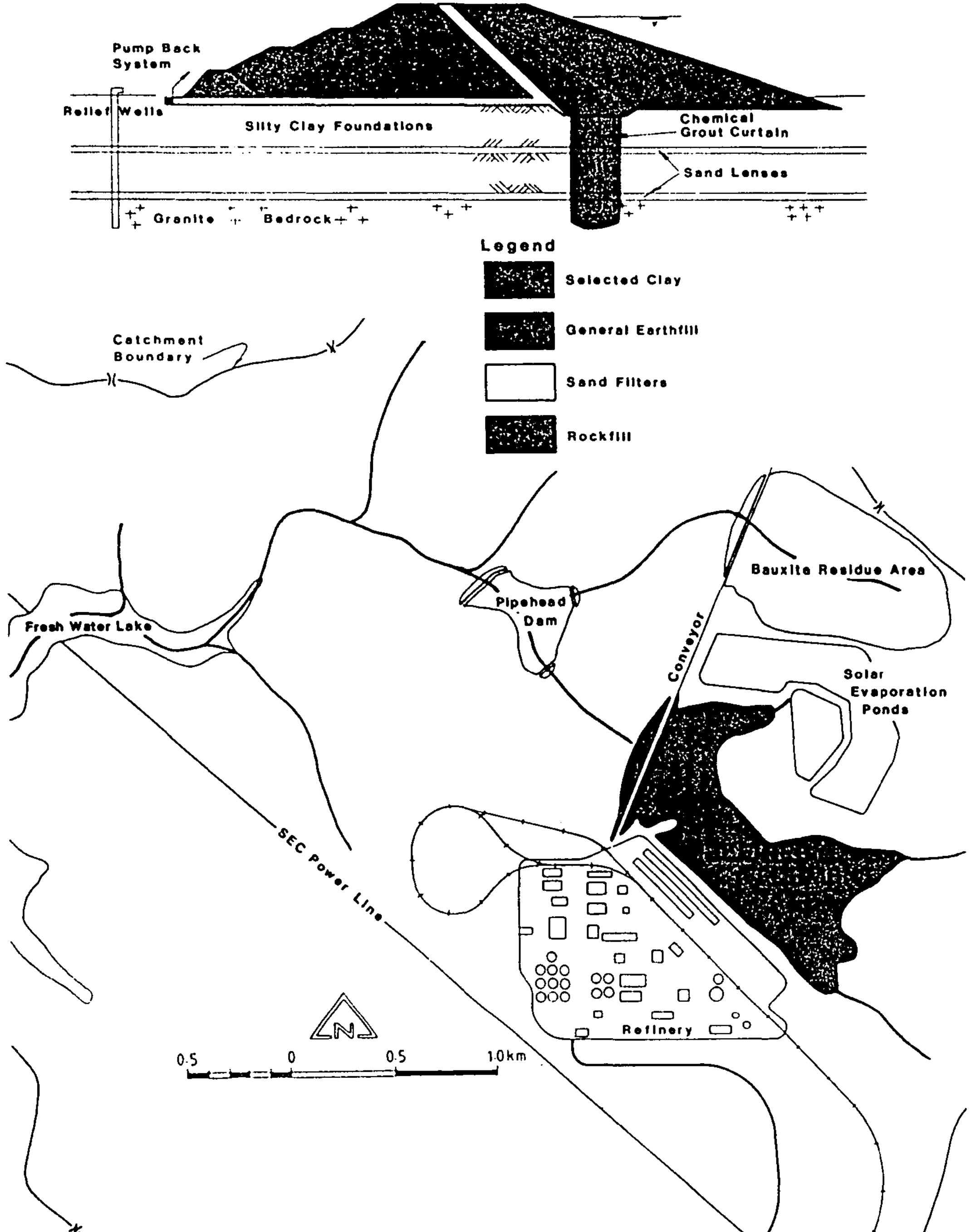
The present location was selected in 1978, having the following advantages:-

- ample fresh water at the site
- deep valleys for efficient storage of residue
- approximately midway between mine and port
- close to an existing railway line
- availability of Collie Coal nearby
- reasonable travel distance for labour from Collie and Bunbury
- reasonable foundation conditions
- located in depleted forest of low quality

Although presently used only by the town of Brunswick Junction, the Augustus-Brunswick River system remains the only major untapped river of high quality in the region south of Perth. Any contamination by caustic compounds from the refinery at the headwaters of this catchment would render the entire river system useless for further development.

An additional problem at the selected site was that evaporation and rainfall are of similar values, hence making it difficult to dispose of surplus contaminated water by evaporation. Precautionary measures to protect groundwater were also essential, as a large part of the river flow is derived from water stored and transmitted through the porous soils which form the upper part of the weathered granite soil profile of the Darling Range.

The value of the water managements works as completed in 1983 was approximately $40 million.



This 28m high earth dam is positioned to trap any contaminated run-off from the 80 ha refinery site, and acts as a holding pond for all caustic process water. With continuing input of chemicals over the years both the dissolved solids and pH will increase with time and it is essential that this water be isolated from the environment.

The dam is unusual in that it has no spillway that could release the caustic water. With rainfall and evaporation of a similar order, the run-off from the refinery and surrounding catchment exceeds the evaporation loss and the dam would overflow within a few years. To minimise run-off, a series of diversion channels surround the lake to carry fresh run-off to the clear streams. A series of test/diversion pits are built into the refinery stormwater system such that major storm flows which may not be contaminated by spillages can be monitored and diverted to clear streams instead of into the catchment lake. To increase water losses by evaporation, the lake also acts as the power station cooling pond using special metal heat exchangers to heat the water.

If the refinery were shut down, the stormwater system would be permanently diverted. The eventual closure of the system at the end of the economic life of the project has been studied and shut down procedures established such that the lake will never spill to the environment.

The foundations of the dam consist of silts, clays and some sand lenses which would result in contaminated underseepage. The more porous soils near the surface were intercepted by a clay filled cut-off trench. Conventional grouting cannot prevent seepage through silts and sands and hence an extensive laboratory and field test programme was instigated to investigate chemical grouting methods. A resin grout, Geoseal MQ4, was eventually adopted as the tests confirmed that it:-

- had a low viscosity approaching that of water, provided temperatures are kept low
- could penetrate both sands and silts
- contained only organic materials and was non polluting
- would not break down under caustic conditions.

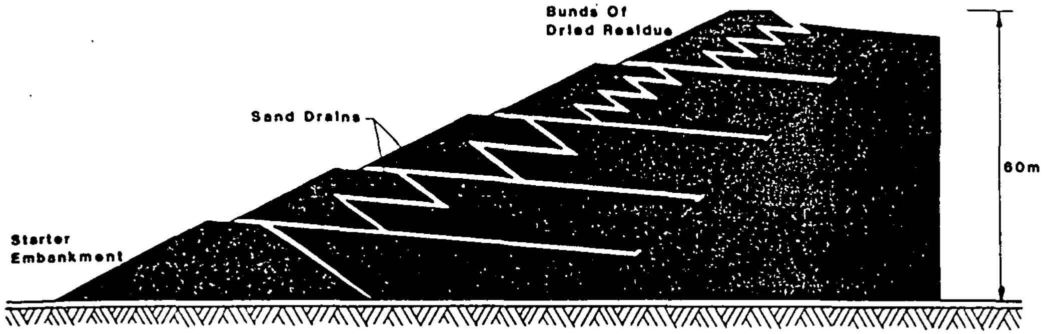
The resulting grouting programme was a significant first in Australian dam engineering, and is described in more detail in the attached technical paper presented at the ANZ Geomechanics Conference.

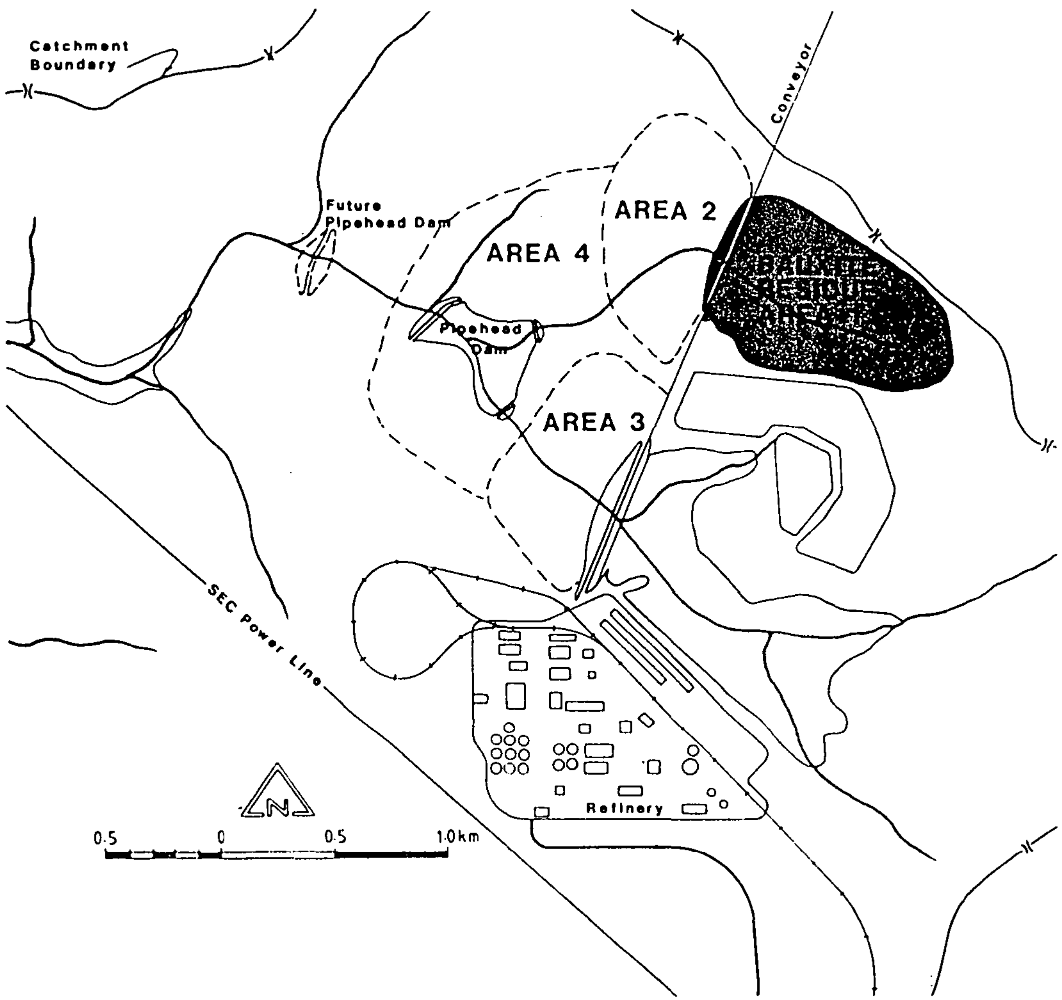
As a second line of defence, a series of monitoring bores was installed at the toe of the dam so that groundwater quality could be checked at any time. These bores are designed to accept a borehole pump to extract any polluted seepage which would be pumped back into the dam. The monitored performance to date shows that such pumping will not be necessary.

STATISTICS

<u>Dam</u>
Construction - zoned earthfill with sand filters
Height - 28m
Length - 800m
Volume of material - 740,000 cubic metres
Contractor - John Holland Constructions
Value of contract - $6M

<u>Grouting</u>
Chemical - Geoseal MQ4 resin grout
No. of grout holes - 750, No. of hook-ups - 25,640
Length of grout holes - 26,700m
Weight of grouts - 369,000 kg
Contractor - Bachy/Grouting and Foundations (WA)
Value of contract - $2.5M





BAUXITE RESIDUE

The quality of bauxite delivered to the Worsley site is low by world standards, and leaves a residue of 2 tonnes for every 1 tonne of alumina produced. For the present 1 Mta plant, this yields 1,600,000 cubic metres of residue annually, for a total of 100 million cubic metres for the presently planned mining life.

The residues are traditionally transported as 25% slurries in water which may contain 20,000 mg/L of caustic chemicals. These slurries decant in the storage to densities typically of the order of 40% solids ie. the contained water is 1 1/2 times the weight or 4 times the volume of the residue solids being stored. Consequently the required volume of storage is large and contains more caustic water than actual solids.

A research programme studied this problem, commencing with the knowledge that it is possible to extract water from the residue on vacuum filters such that the material as delivered to storage has 65% solids ie. the contained water is 1/2 the weight or 1 1/2 times the volume of solids. The research programme which was initially carried out in Arkansas, USA and completed in Australia showed that it was possible to pump filtered material at up to 65% solids. The residue was proven to essentially behave as a silty soil, having low strength when saturated, but when dried having sufficient strength to be quite stable and suitable for building conventional earth structures.

The filtered residue scheme was shown to have the following advantages:-

- the low volume of water accompanying the residue results in less caustic water being exposed to the environment
- the increased density results in a lower storage volume, decreasing the area covered by the residue
- it is possible and economically attractive to wash the residue on the filters, recovering valuable compounds and reducing the caustic level of water to the environment to 8000 mg/L.

Research and analysis showed that even this filtered residue would decant some water, and in summer the residue would be dried to a soil like consistency such that it could be trafficked and handled with earthmoving equipment. It is therefore possible to commence storage behind a low cost starter bund and to use dried residue from time to time to build additional bunds to raise the storage. The embankment to carry the bauxite conveyor has been specifically designed to serve a double purpose in acting as the first starter bund.

LETTERS PATENT

AUSTRALIA

STANDARD PATENT

ELIZABETH THE SECOND, by the Grace of God Queen of Australia and Her other Realms and Territories, Head of the Commonwealth.

To all to whom these presents shall come Greeting:

WE DO, by these Letters Patent, give and grant to the person whose name is specified hereunder Our Special Licence and the exclusive right, subject to the laws in force from time to time in Australia or a part of Australia, by itself, its agents and licensees, at all times during the term of these Letters Patent, to make, use, exercise and vend throughout Australia the invention the title of which is specified hereunder and being the invention that is fully defined in the claim or claims of the complete specification accepted in accordance with the Patents Act 1952 in such manner as it thinks fit, so that it shall have and enjoy the whole profit and advantage accruing by reason of the invention during that term.

Name(s) of Patentee(s): GHD-DWYER (WA) PTY. LTD.

Address of Patentee(s): 7 HARDY STREET, SOUTH PERTH, WESTERN AUSTRALIA, AUSTRALIA

Name(s) of Actual Inventor(s): JOHN T. PHILLIPS

Title of Invention: DISPOSAL OF FINE TAILINGS

Number of Complete Specification: 530388

Term of Letters Patent: Sixteen years commencing on 3 December 1982

IN WITNESS whereof our Commissioner of Patents has caused these Our Letters Patent to be dated as of the 3 December 1982, and to be sealed with the seal of the Patent Office on 10 November 1983.

F.J. SMITH
Commissioner of Patents

Stability analyses demonstrated that the pore pressures within the wet winter slurry deposits would quickly build up to unacceptable values, and hence sand drains are to be installed to enable the dissipation of these pressures. These drains form a zone of rapidly consolidating residue behind the thin face of constructed bunds, thereby creating a strong zone of material which adequately ensures the stability of the entire mass even though the material further back may still be of slurry consistency. The volume of earthmoving to achieve stability is thereby only a small portion of the total volume of stable material.

The strength of the bunds and rapidly consolidated material enables the residue to be economically stacked to 65 metres height even under earthquake conditions. This is approximately twice the height of previous residue systems and results in an even smaller area of land being covered by the caustic residue.

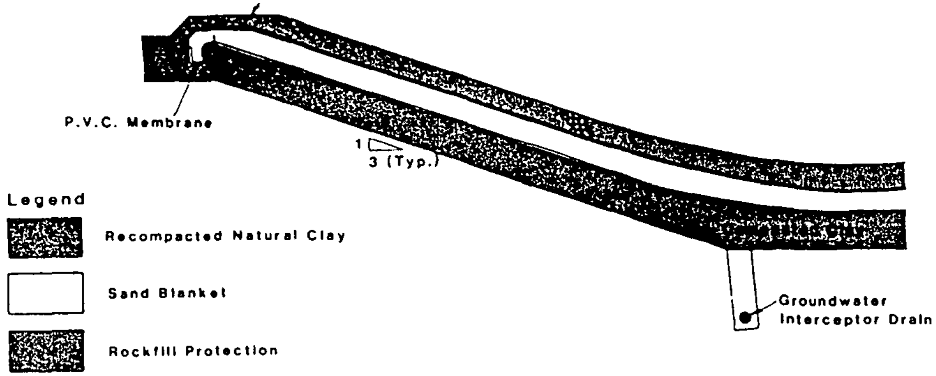
To prevent the caustic water from penetrating into the groundwater, the foundations were prepared with drainage and sealing layers. The permeable upper level of soils were screened into sand and gravel fractions, the underlying silty clays moistened and compacted to form a seal and controlled drainage layers of sand and gravel replaced. The drainage layers were intersected with slotted pipe drains which drained at atmospheric pressure to the pipehead dam from where the water is reclaimed to the catchment lake. The complete layer under the residue is therefore at atmospheric pressure, thus removing any pressure that would force seepage through the seal into the underlying groundwater.

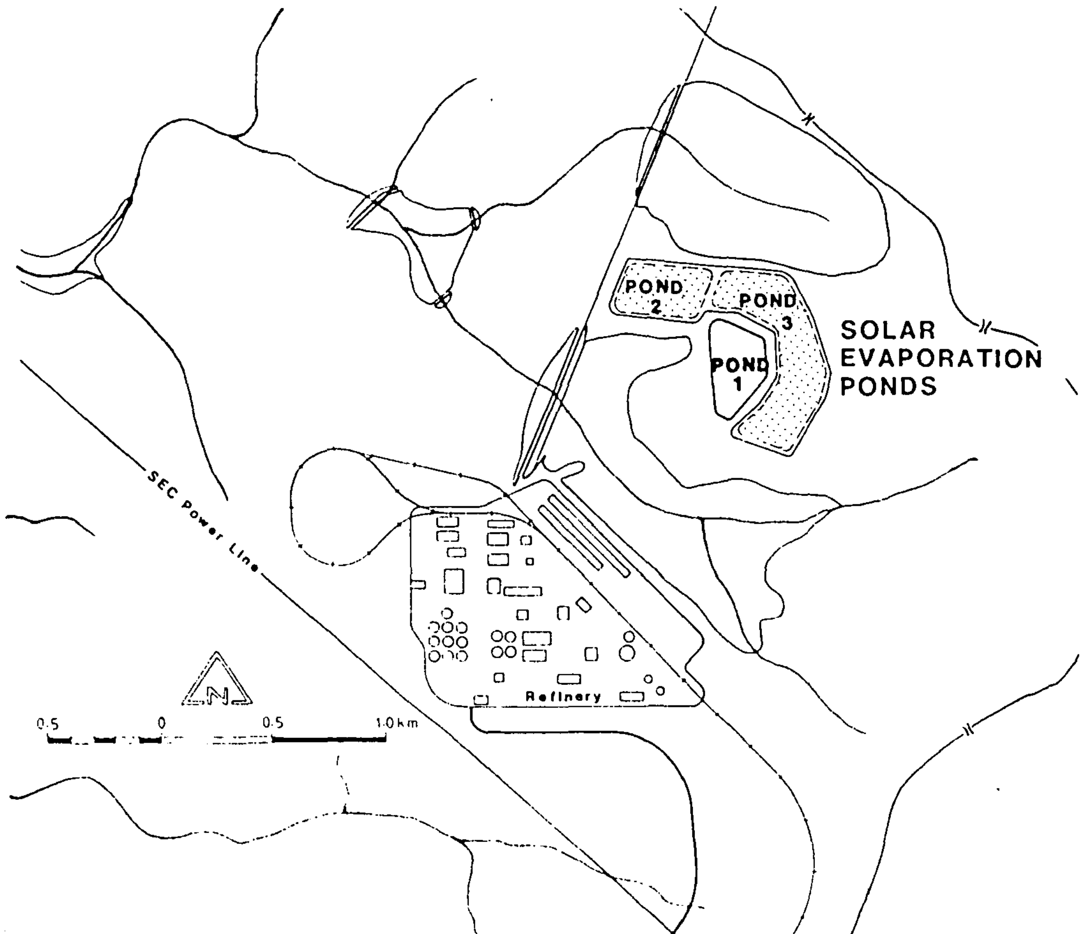
Decanted water and contaminated rainfall run-off are directed through decant towers to the pipehead dam, and are pumped from there to the catchment lake. As one area is filled to capacity, another area will be developed. Each area is deliberately restricted in size to minimize the amount of rainfall runoff from polluted areas. A completed stack of residue will be contoured, the upper layers drained and revegetated, with trials currently in progress to confirm the optimum methods.

The system of handling residue has been termed a "dry residue" system, and has many significant cost and environmental advantages over previous systems. The system is a significant step forward in the handling of such toxic residues, and has been accepted as a new invention and is protected by patents in both Australia and the USA.

STATISTICS

Area of first residue area	66 ha
Planned max height of first residue stack	40 metres
Volume in first stack	10,000,000 cubic metres
Total earthworks to first area	800,000 cubic metres
Contractor	F.K. Kanny & Sons
Value of contract	4.5 million





SOLAR EVAPORATION POND

The cleaning processes in the refinery use sulphuric acid and toxic chemicals which would be deleterious to the process it placed in the catchment lake. These will be disposed and stored in sealed evaporation ponds which are located on a ridge between the catchment lake and first residue area. In this manner all the potential polluting areas are contained in a single compact area, leading to more efficient deployment of protection and monitoring systems.

Due to the small margin of evaporation over rainfall, the ponds do not truly dispose of the material, since at low pond levels there is additional run-off from the pond sides which exceeds evaporation, thereby increasing the level until the ponds become self balancing when nearly full. Sufficient area has been allowed to store the relatively small volumes of these materials as they accumulate over the years.

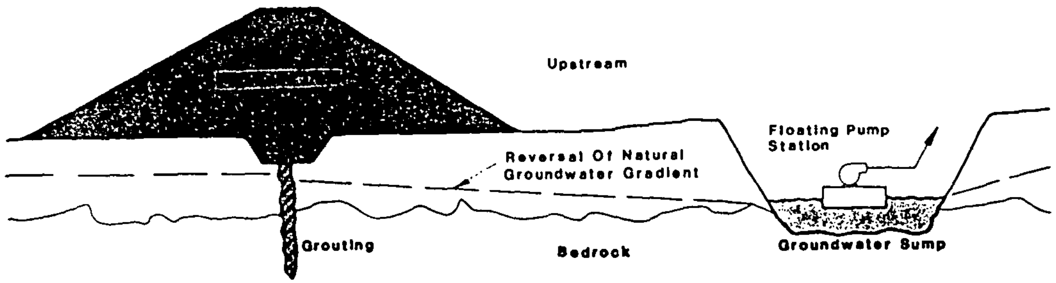
At the end of the economic life of the plant the ponds are self balancing. Options available for further rehabilitation include recovery of chemicals, enhanced evaporation by sprays similar to many liquid disposal sites, or neutralisation in the abandoned catchment lake.

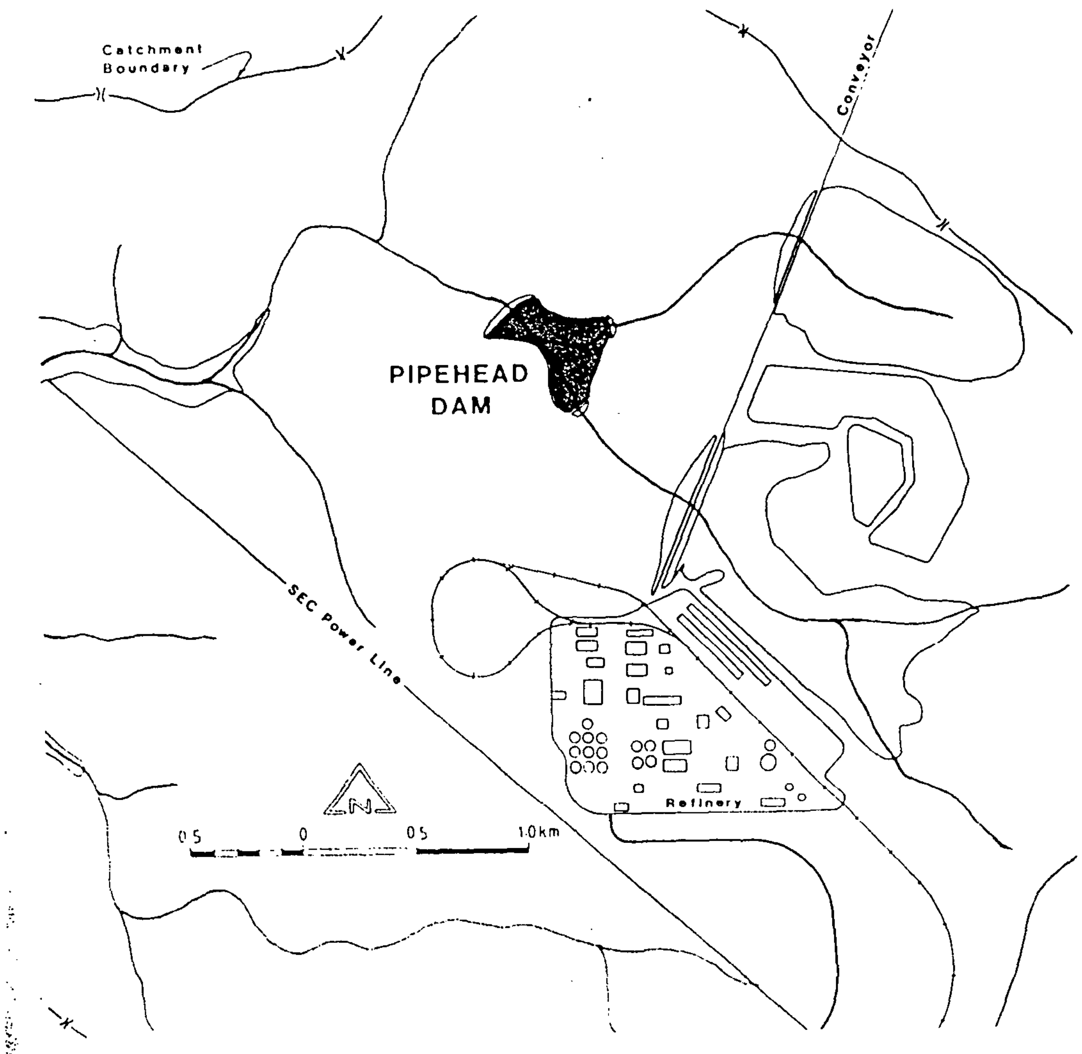
The ponds are lined with PVC material which is protected from UV light deterioration by layers of sand with rip rap at high levels. Calculations showed that if the PVC were laid upon a sand bed in the usual fashion, any leaks from an inadvertent puncture or leaking seam would spread in an unacceptable fashion, and hence the PVC was laid directly on compacted clay which formed a second sealing barrier. The reliability of this double system is several orders higher than the conventional system.

Most PVC materials suffer loss of plasticity when in contact with clay, and hence an exhaustive series of tests and chemical analyses were carried out to find a formulation which could provide the appropriate surety of material performance.

STATISTICS

Total area of ponds	39 ha
Area of PVC in first pond	78000 sq. metres
Storage volume of first pond	440 ML
Construction. PVC, 0.7mm thick on compacted clay, protected by sand and rip rap	
Contractors - John Holland Construction and Staff Engineered Membranes	
Value of contracts	$5.5 million





The initial concept of the pipehead dam was to collect only polluted underdrainage, decant and rainfall run-off from the residue areas. It was to be sized to accommodate storm flows and water would be pumped back into the refinery catchment lake.

A site was found at the confluence of two streams such that the dam was strategically located below all the areas which contain potential sources of pollutants. The dam then retains its original purpose but can be quickly adapted to collect any leakage or seepage from upstream areas. In normal operation all run-off from adjacent catchments is diverted around the dam to the clear streams.

The pipehead dam has no spillway and can hold the flow from large storms with ample freeboard even if the pumps did not operate. It normally operates in the empty condition, requiring special design features to prevent shrinkage cracking of the dam core.

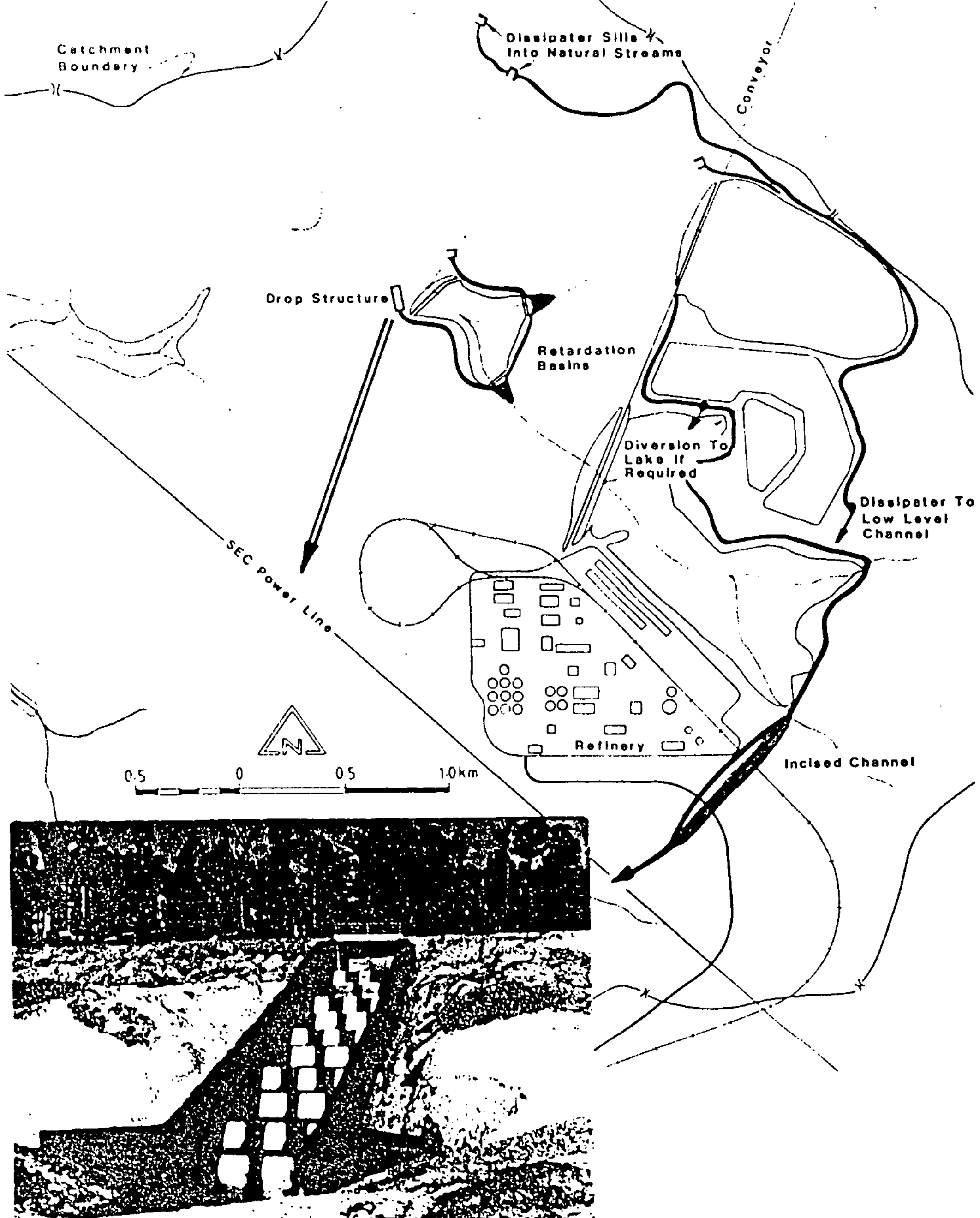
Since the dam is empty at most times, the design was modified to collect all groundwater in the area, thereby providing assurance that any seepage that might bypass the protective systems upstream would be intercepted at this point. A sump was excavated into the rock behind the dam and the water level is held down below groundwater level in the sump. Two boreholes adjacent to the sump ensure that the groundwater lowering effect is maintained across the valley.

As a precautionary measure in times of flood, the dam is designed as a watertight structure and is sealed against underseepage by grouting into the underlying rock. Groundwater conditions are monitored downstream of the dam in a series of bores that are designed to accept a pump to extract any polluted groundwater that might arise under an extreme circumstance.

This triple system of protection will be maintained as the residue areas cover the catchments upstream. Eventually the dam will be engulfed by residue, by which time a similar dam will be installed downstream. The present location is the most cost effective in both capital costs and ensures the most effective diversion system to carry fresh water around the dam.

STATISTICS

Dam height	15m
Dam length	270m
Volume of earthworks	160,000 cubic metres
Contractor	Thiess Bros
Contract price	$3 million



CHANNELS

The various diversion channels serve two main purposes:-

- to divert fresh water away from polluted areas to assist the water balance of those areas and to maximise fresh water flow downstream

- to collect any polluted water from contaminated residue areas and divert this into an appropriate safe storage.

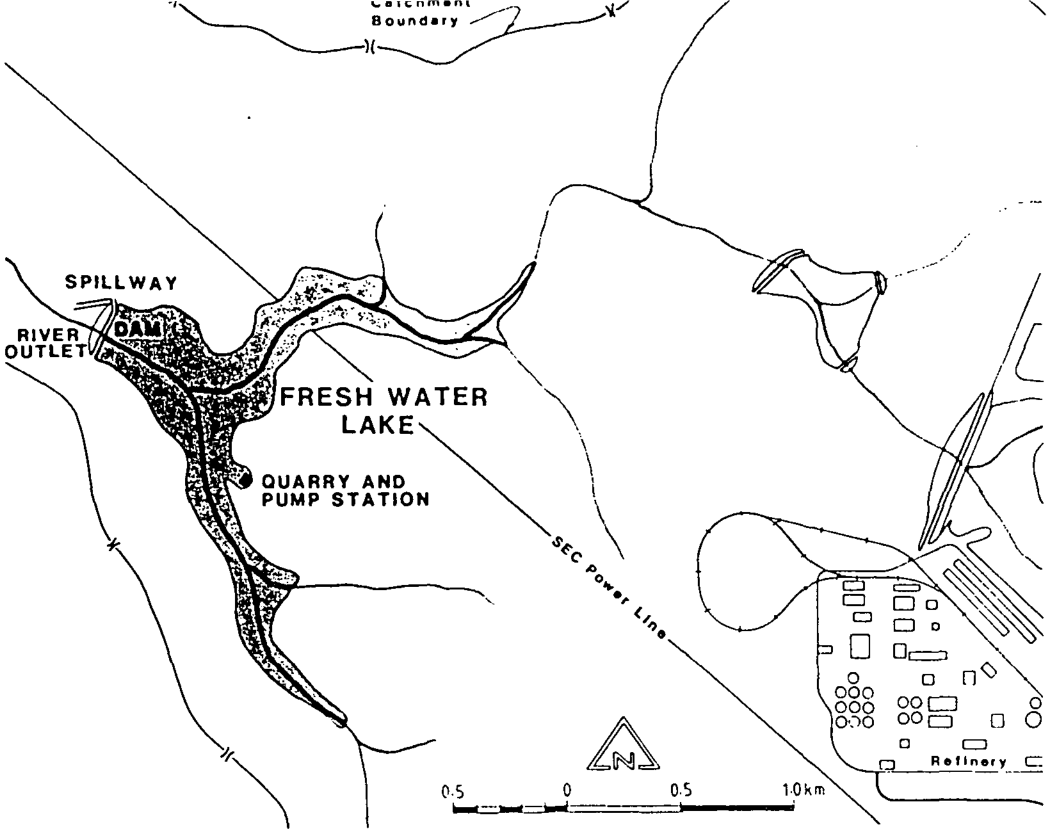
The channel system indicated on the figure is shown only at the first stage of development. As the residue rises above the valley sides, run-off from the berms may be contaminated and the channels that originally carried fresh water may carry polluted water. Similarly, as additional land is covered by residue then previously fresh run-off should no longer be diverted to fresh streams, and the water should be diverted to, say, the pipehead dam. The complete scheme has been modelled on a year by year basis for 60 years, and the channels have been positioned and designed for maximum useful life and ease of conversion from fresh to polluted or vice versa as time proceeds.

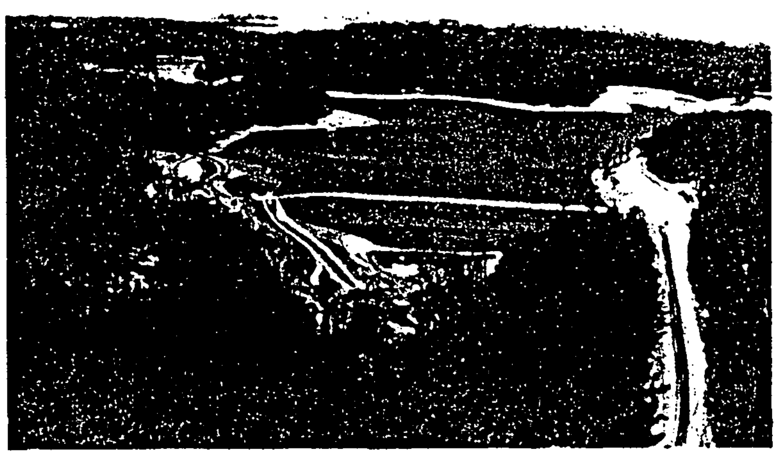
In most places the channels are simple earth structures with appropriate erosion control structures, particularly where intercepting larger creeks. Depths are sufficient to intercept any groundwater flowing through the more permeable upper soil horizons, and in some places the base of the channels are clay lined where they cross more permeable ground.

Where possible the channels enter natural stream beds via cascades of naturally occurring rock materials. The drop of the channel around the pipehead dam did not warrant full spillway construction, and energy dissipation was economically achieved by the novel rock gabion/concrete teeth arrangement as shown. Construction of the channels was incorporated in the contracts for nearby dams as appropriate.

STATISTICS

Length of channels, Stage 1	21 km
Combined design flow capacity (100 yr)	54 cumecs
Value of works	$2 million





FRESH WATER LAKE DAM

An alumina refinery requires a good supply of high quality water, and this is supplied by the Fresh Water Lake Dam (FWL). The dam was positioned below the confluence of the two major streams within the Worsley lease, hence collecting and controlling all water within the lease boundaries. The site of the dam was a balance between economics and the need to minimise clearing of good quality forest in the lower reaches of the stream.

The left abutment of the dam consisted of weathered materials up to 30m deep, whilst the right abutment was underlain at shallow depth by sound rock. Under the weight of the dam the left abutment will settle, potentially creating a crack through the dam core as that portion of the dam settles.

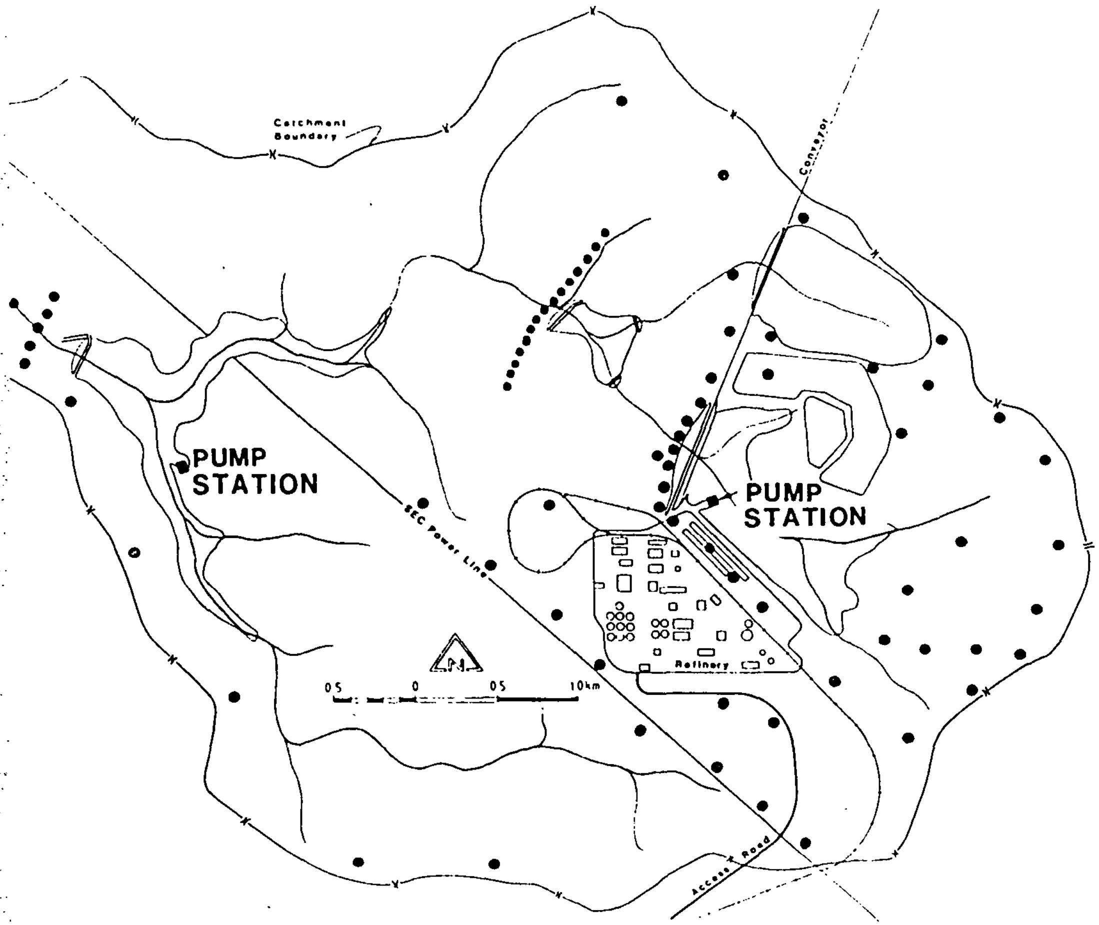
Design of the dam was of homogeneous earth with clay filled cut-off trench, sand chimney drain and protective rip rap. The foundation areas and potential cracking areas were constructed of carefully selected high plasticity materials and the performance of the dam, particularly in the danger areas, has been carefully monitored during construction and filling without detecting any signs of undue stress above those predicted by finite element analyses.

The dam contains a buried conduit, used as a river diversion during construction but designed to release water in a controlled manner for downstream users. The concrete lined spillway was built over relatively soft ground and incorporates anchor bars into earth to withstand uplift and has an unusual continuously formed main chute so that there is no risk of erosion from a partly open construction joint.

A quarry in a dolerite dyke in the upstream portion of the river provided rock for rip rap and crushed stone. The position and form of the quarry was adjusted to form a sump for a 30m high pump station structure, allowing simple extraction of water over all operating levels at a point close to the refinery.

STATISTICS

Height of dam	33m
Length of dam	350m
Volume of earthworks	390,000 cubic metres
Volume of storage	6000 ML
Spillway capacity	30 cumecs
Main contractor	Citra Constructions/ Thiess Bros
Value of contracts	$11 million



GROUNDWATER MONITORING AND MISCELLANEOUS

At an early stage of investigations a network of groundwater monitoring bores were established. These have been regularly monitored and sampled to provide an understanding of natural groundwater movements, variation with seasons, water quality and the natural levels of pH and heavy metal traces. Selected bores have been fitted with automatic continuous recording instruments.

As designs developed and the works were constructed, additional bores were placed below crucial structures or in strategic positions to monitor specific areas where there was a potential for groundwater pollution. In most cases these bores are showing groundwater patterns consistent with natural or predicted design values.

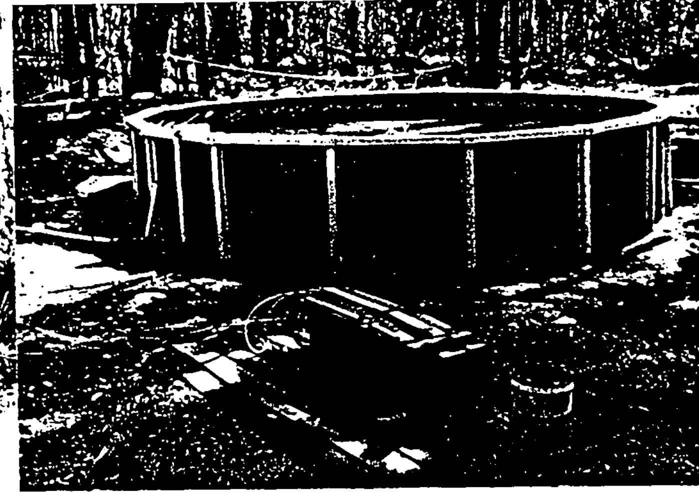
Two major pump station structures were designed, one for fresh water supply and the other for power station cooling/process water supply with a capacity of 7000 L/sec. A floating pump station was designed for the pipehead dam with an articulated steel truss of triangular form holding the pump station in position as it rose and fell over the wide range of possible water levels.

Other works included two coffer dams for construction purposes, a stream gauging weir and station, spillway modelling, building of a model of the entire water management system, bridges over the main spillway and other channels, and miscellaneous earthworks/civil works.

GHD-Dwyer provided technical supervision during construction, field engineers and supervisors, specialist construction advice, reading of dam instrumentation during and after construction and preparation of operation/maintenance manuals.

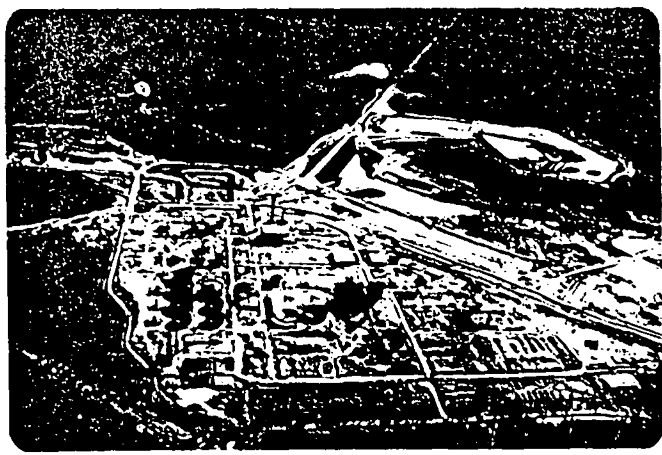
	STATISTICS
Number of monitor holes	73
Length of holes drilled	2500m
Cost of monitoring system	$0.9 million
Civil/Structural cost of pump stations	$1.2 million
Engineering, investigation and research costs	$3 million

THE FOREST, THOUGH DEPLETED OF GOOD TIMBER, STILL HAS AN ATTRACTION OF ITS OWN.



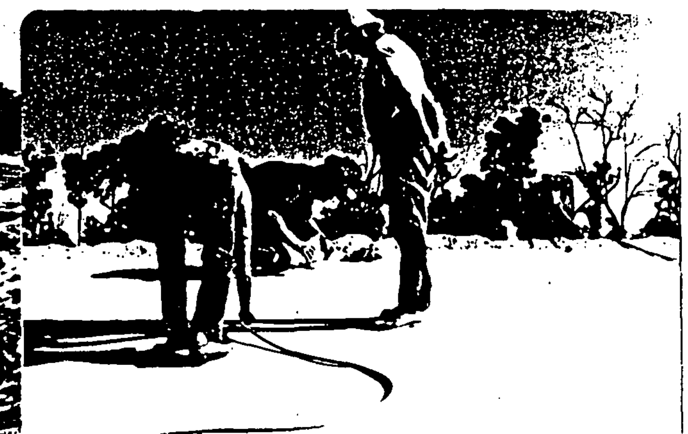
LARGE SCALE PERMEABILITY TEST OF HIGH POROSITY SURFACE SOILS IN BAUXITE RESIDUE AREA.

PREPARATION OF BAUXITE RESIDUE AREA WITH SAND DRAIN LAYERS AND BURIED DRAINS.

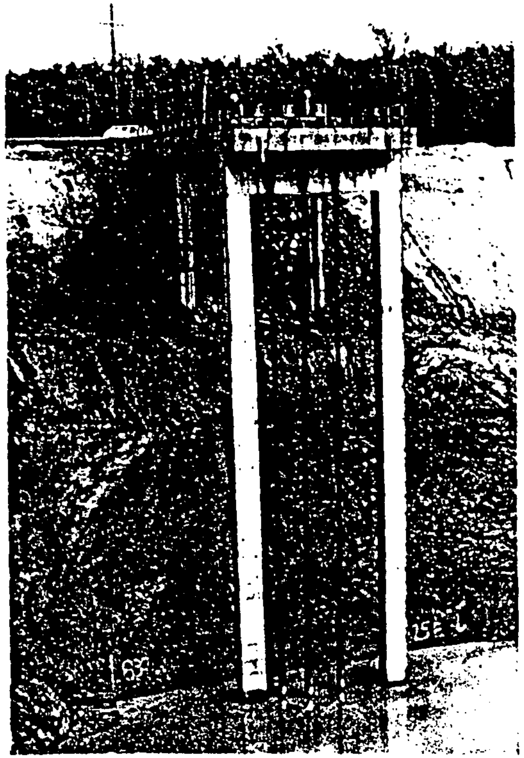


REFINERY IN FOREGROUND DRAINS TO REFINERY CATCHMENT LAKE AT REAR. SOLAR EVAPORATION PONDS UNDER CONSTRUCTION ON RIGHT, CLEARING FOR PIPEHEAD DAM AT LEFT, CONVEYOR LINE STRETCHES INTO THE DISTANCE.

COVERING THE PVC LINER WITH PROTECTIVE SAND IN SOLAR EVAPORATION POND.



JOINTING AND TESTING OF PVC LINER TO SOLAR EVAPORATION POND.



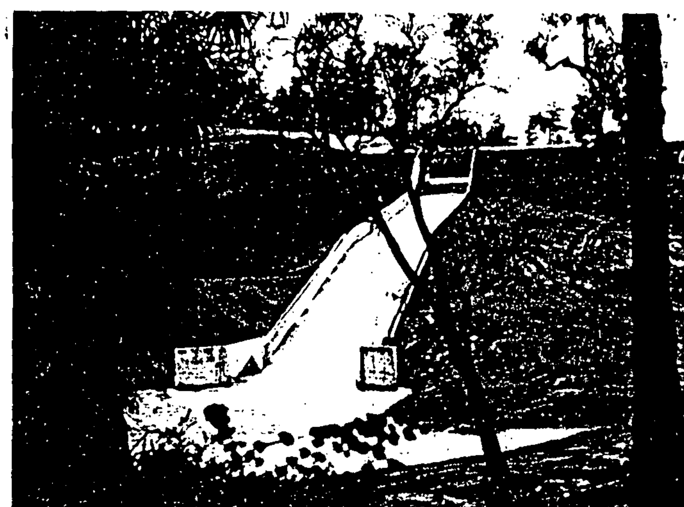
PUMP STATION MOUNTED FROM SIDE OF QUARRY IN FRESH WATER LAKE. THREE VERTICAL TURBINE PUMPS.

COOL WATER INTAKE & PUMP STATION IN REFINERY CATCHMENT LAKE.

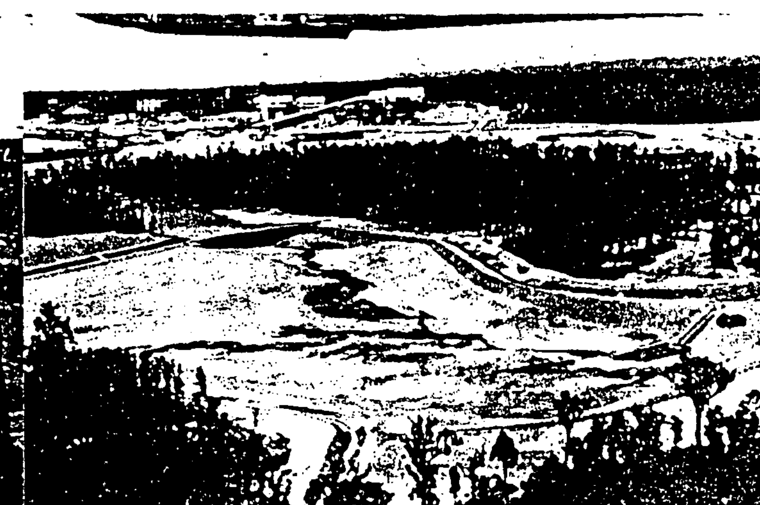
CONTAINS 8 VERTICAL TURBINE PUMPS.

PIPEHEAD DAM WITH BYPASS CHANNEL AND DISSIPATOR AT REAR.

FLOATING PUMP STATION IN POND.



SPILLWAY TO FRESH WATER LAKE. FLOOR OF CHUTE IS CONTINUOUS CONCRETE WITHOUT JOINTS.



PIPEHEAD DAM ISOLATED FROM SURROUNDING CATCHMENTS BY CHANNELS. FLOOD SURGE POND AT REAR. GROUNDWATER BEING COLLECTED AS IT SEEPS INTO STORAGE.

Appendix B - Brett, D.M., and Osbourne, T.R., Chemical Grouting of Dam Foundations in Residual Laterite Soils of the Darling Range, Western Australia. , 4th ANZ Conference on Geomechanics, Perth, 1984.

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Appendix C - Brett, D.M., Grouting of Low Permeability Soils Using Tube a Manchette Techniques, Proceedings, Conference of the National Waterwell and Drilling Association, Bunbury, Western Australia 1982.

Appendix D - Brett, D.M., Chemical Grouting for Dams, presented at the ANCOLD Study Tour, N.S.W., 1985. To be published ANCOLD BULLETIN 1986.

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Appendix E - Truscott, E.G. and Brett, D.M., The Dams of the Worsley Project, ANCOLD Bulletin, April 1984.